CIVIL ENGINEERING

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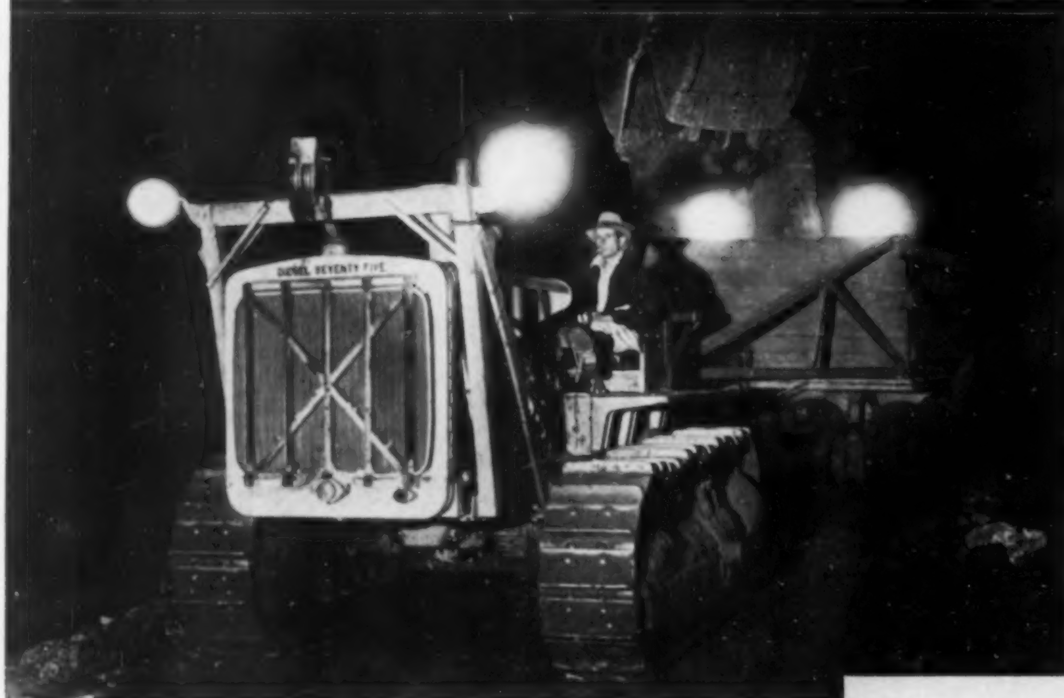
Cut-Off Wall of Concrete Caissons for Dike at Quabbin Reservoir
Metropolitan District Water Supply for Boston and Environs

Volume 4 — AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852 — *Number 6*

JUNE 1934



ABOVE

At 11,500 feet altitude, a "Caterpillar" Diesel Fifty Tractor hauls supplies for a Colorado water diversion tunnel job, clearing its own way through snow. Fuel cost is only $1.80 per 8-hour shift.

LEFT

Night and day, this "Caterpillar" Diesel Seventy-Five Tractor hauls 40-ton loads of rock and gravel—low cost material transportation for the construction of the San Francisco-Oakland Bridge.

BELOW

Fuel costs for road work in Upsher County, Texas, were cut from $3.50 to 40 cents per day when this "Caterpillar" Diesel Thirty-Five Tractor went to work.

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• • • says a Nebraska contractor who is doing $16.66 more work per day at $7.20 less fuel cost than with his former gasoline tractor.

Fuel economy is the feature every owner lists first among the "Caterpillar" Diesel's advantages—the economy of lower price fuel plus lower fuel consumption. Many users find, too, that the "Caterpillar" Diesel with its steady, lugging power does more work per day than their former gasoline tractors. And all owners praise this tractor's dependability, easy and sure starting, simple operation and maintenance. Today, there are more than 2500 users of "Caterpillar" Diesels. There are three tractor sizes available, and three sizes of stationary power units. Caterpillar Tractor Co., Peoria, Illinois, U. S. A.

• AMERICA GOES DIESEL •

Among Our Writers

Frank E. Winsor, a graduate of Brown University, from which he received the honorary degree of doctor of science in 1929, was engaged for several years on the Metropolitan Sewerage System of Boston, and later on the Metropolitan Water Works of Boston and the Charles River Basin. He was a member of the Catskill Aqueduct organization for about nine years—from 1910 to 1915 as department engineer in the Southern Aqueduct Department, including the Kensico and Hill View reservoirs and about 30 miles of the Catskill Aqueduct. He was chief engineer on the new water supply for Providence, R.I., from 1915 to 1926, since which time he has held his present position.

Irwin E. Burks, from 1923 to 1926, as resident engineer in charge of concrete pavement projects for Santa Fe, N.Mex., purchased modern testing equipment and organized a state program of tests and inspection—one of the first of its kind. During the next five years he was chief concrete technician for the Aluminum Company of America, supervising all concrete operations on the Santeetlah, Calderwood, and Chute à Caron dams. In 1931 he was concrete superintendent for the Thomas W. Koon Dam and had supervision of field operations and direction of the laboratory. His connection with Madden Dam dates from 1931.

H. G. Smith, after two years as chief draftsman for an Alaskan railway, became division engineer of the Oregon Short Line Railroad. He then served successively as office engineer, City Engineer's Office, Salt Lake City; designing draftsman, City Engineer's Office, Los Angeles; major of engineers, U. S. Army; and construction engineer and resident engineer, Union Iron Company and Union Oil Company. Since 1924 he has been engineer of sewer design for the City of Los Angeles.

Lynn Perry has been a frequent contributor to the publications of the Society. He is a graduate of the University of Pennsylvania and since 1919 has been Assistant Professor of Civil Engineering at Lafayette College, Easton, Pa., and in charge of hydraulic and sanitary engineering courses. History is one of his hobbies, and he has written several short historical papers.

Raymond P. Pennoyer's 20 years of service with the Carnegie Steel Company include twelve years in the Homestead Mills. This work involved much operating experience and supervision of a wide variety of construction work. For the past six years he has been in direct charge of sheet-piling engineering and sales.

H. A. Van Norman began work for the City of Los Angeles in 1906 on the Owens River Aqueduct. He had charge of the building of the power plants in the Owens Valley Division and then took charge of the construction of the Mojave Division and later directed the completion of the construction of all the divisions. In 1923, as chief engineer, he built the Hyperion Outfall Sewer for the city and then became city engineer. In 1925 he returned to the Department of Water and Power as assistant chief engineer and general manager of the Bureau of Water Works and Supply. He has been chief engineer since 1928.

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NUMBER 6

Boston's New Metropolitan Water Supply

Expansion of Historic System Requires Solution of Legal, Hydraulic, and Construction Problems

By FRANK E. WINSOR
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF ENGINEER, METROPOLITAN DISTRICT WATER SUPPLY COMMISSION, COMMONWEALTH OF MASSACHUSETTS

ONE of the important water supply improvements of the United States has been in process of construction in Massachusetts since 1926. This account, by the chief engineer, is a résumé of the absorbing story told before the Society's Metropolitan Section on February 21, 1934. Starting with the now historic Connecticut River diversion case, Mr. Winsor brings the work down to date. The 25-mile tunnel joining the older Wachusett system is rapidly progressing as a result of good engineering design and efficient construction. Of particular interest will be found his description of the spiral-lined shaft 260 ft deep, capable of diverting eventually two billion gallons daily from the Ware River. Still another feature that makes the work memorable is the design and construction of two large earth dams, particularly as regards their articulated concrete core walls sunk in 130 ft of ground water. These dams when completed, about 1940, will form the Quabbin Reservoir. This account is a notable review of an important job.

BESIDES being the fourth most populous center in the country, the Metropolitan District of Boston probably has more political units than any other. Within ten miles of the Massachusetts State Capitol there are 35 cities and towns, each having its own separate government, elected officials, and revenues. Boston itself contributes only 800,000 to the total population of 1,900,000 in this district.

Beginning in 1889, there has gradually grown up a system of regional control of certain public works, both as regards construction and operation. These works, which since 1919 have been in charge of the Metropolitan District Commission, include water supply and sewerage systems and parks, but they do not include

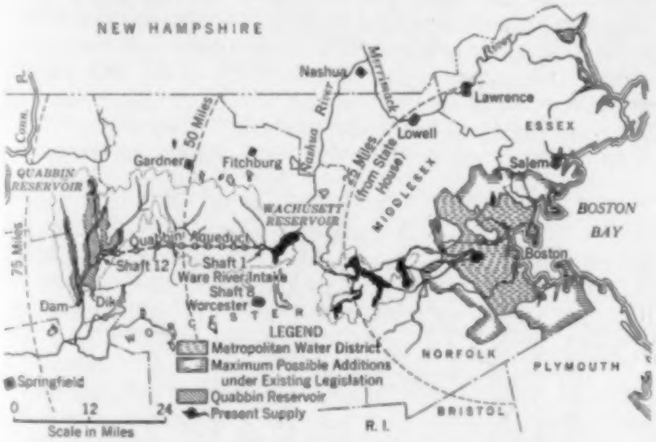


FIG. 1. QUABBIN RESERVOIR AND AQUEDUCT
Under Construction by the Metropolitan Water District

distribution systems, local sewers, or local parks. Although in each of these enterprises the commonwealth, that is the state, advances credit for the work, the bills are eventually paid by the communities benefited. For example, in the case of a water supply system, each community is required to set aside, in its annual tax budget, a sum sufficient to defray its proportionate share of the operation and maintenance of the system, the interest and annuities on bonds, and any other incidental expenses.

As Boston was one of the first American cities to develop a modern system of water supply, its history in this field is fairly well known. First Jamaica Pond was used as a source of supply. In 1848 the system was expanded to include Lake Cochituate. In 1876 the Sudbury supply was added, and in 1908 the Wachusett Reservoir. In 1915, as a result of the general introduction of meters, consumption was reduced from 130 gal per capita daily to 88 gal. By 1918, however, it was evident that, in spite of this reduction, the existing sources would soon be inadequate. After about eight years, legislation was procured under which the present major addition to the supply is being constructed.

As shown in Fig. 1, the new works lie to the west of the Wachusett Reservoir and tap the Ware and Swift rivers. To the extreme west is the Quabbin Reservoir, the largest and the final addition to be made to the system. Water will thus travel a maximum distance of about 65 miles to supply the needs of Metropolitan Boston. Aqueducts, both old and new, will comprise about 50 miles of this distance.

CONNECTICUT OBJECTS

No sooner had Massachusetts passed state legislation permitting the Metropolitan District to tap the Ware and Swift rivers than public officials in Connecticut began to consider the result of allowing any diversion of interstate waters. The Ware, the Swift, and the Quaboag join at a point known as Three Rivers to form the

Chicopee River, which flows into the Connecticut. The State of Connecticut took the view that Massachusetts had no right to take any water that would otherwise reach the Connecticut River and flow through the State of Connecticut. The same question had been raised before.

This case—together with one very much like it, be-

WARE RIVER DIVERSION DAM AND INTAKE UNDER CONSTRUCTION
Openings to Siphon Spillways Shown in Foundation of Gate House

tween the states of New York, New Jersey, and Pennsylvania, whereby New Jersey sought to prevent the diversion of water from the Delaware River for the water supply of the City of New York—has established some interesting and fundamental principles. Connecticut based its bill of complaint on several grounds: effect of reduction in stream flow on navigation; reduction of water power; damage to agriculture; increase of pollution in the river due to decreased upland flow; injury to fish life; and, finally, lack of necessity for the Metropolitan District to divert these interstate waters, since it was claimed that there were supplies of ample quantity in the eastern part of Massachusetts that should be used.

Following hearings before a master appointed by the U. S. Supreme Court, a decision was rendered after about two and a half years. Under this decision Massachusetts is permitted to take about 93 per cent of the amount of water it might have taken under its own legislation, the modification being prescribed by the U. S. War Department to prevent the reduction of the navigable depth of the Connecticut River in times of low flow. Less than 3 per cent of the flow of the Connecticut River will be diverted by the development, and the greater part of the diversion will occur at times of high stream flow.

In its decision, the Supreme Court established several principles, the most important of which are that there may be a diversion of interstate waters, and that establishing a domestic water supply is the highest use to which water can be put. It further recognized the right of a community to select an unpolluted, rather than a polluted source of supply. The eastern part of Massachusetts is very thickly populated, and the streams in that area are polluted.

On the Swift River, 25 miles west of Wachusett, lies the site of the Quabbin Reservoir, which will be very large for this section of the country. Although small compared with the reservoir to be formed behind Boulder Dam, it will contain about 415 billion gallons (1,274,000 acre-ft), or three times as much water as the Ashokan Reservoir. It will have 151 miles of shore line and will contain 60 islands, one of which will be over $3^1/_2$ miles long and a mile wide.

It is the policy of the Metropolitan District Water Supply Commission to acquire a liberal amount of land around the reservoir for sanitary protection. As a rule, this land is not expensive. Some 57,000 acres have already been bought, and a total of more than 100 sq miles will probably be acquired. The main dam will be 170 ft high above the river bed and will extend down at least 110 ft to bedrock. A second dam, or dike, will be about 135 ft high, plus a 129-ft cut-off down to rock. The site of the dike appears to have been the location of the Swift River in preglacial times.

Quabbin Reservoir, which has an area of 25,000 acres, will flood about 16 miles of single-track railroad and about 8 miles of high-tension transmission line. The latter has already been relocated. The site is ideal for a reservoir, as the ground is sandy, with negligible areas of swamp, and the average depth of water will be in excess of 50 ft. There are several small mills in the area, but active manufacturing had practically ceased before the project was authorized. It will be necessary to move 16 cemeteries with between 5,000 and 6,000 bodies, about 1,400 of which have already been removed. A new cemetery has been established. Three entire towns will be wiped out of existence. By the act authorizing construction of the reservoir, their remnants will be annexed to other towns in order to provide for ordinary civil and criminal processes. County and town lines will be very materially changed.

AQUEDUCT SERVES VARIOUS USES

Leading east from the reservoir is the tunnel known as the Quabbin Aqueduct, which will be nearly 25 miles long. This will perform a number of rather unusual functions. As indicated in the profile, Fig. 2, the Wachusett Reservoir to the east is 395 ft above sea level, and the Quabbin Reservoir will be 135 ft higher, at an elevation of 530 ft. The weirs at Shaft 8 are at an elevation of 656 ft, so that water taken in there may be sent through the Quabbin Aqueduct either to the Wachusett or to the Quabbin Reservoir. The flow through the tunnel will be controlled at the Wachusett end. The reason for the depressed section at the east end is to keep the tunnel in sound rock and to enable the water to be brought up through a shaft under pressure to a prospective power plant at Shaft 1. This will utilize whatever power there is in the 135 ft of maximum fall between the Quabbin and Wachusett reservoirs.

At Shaft 8, water from the Ware River can be taken

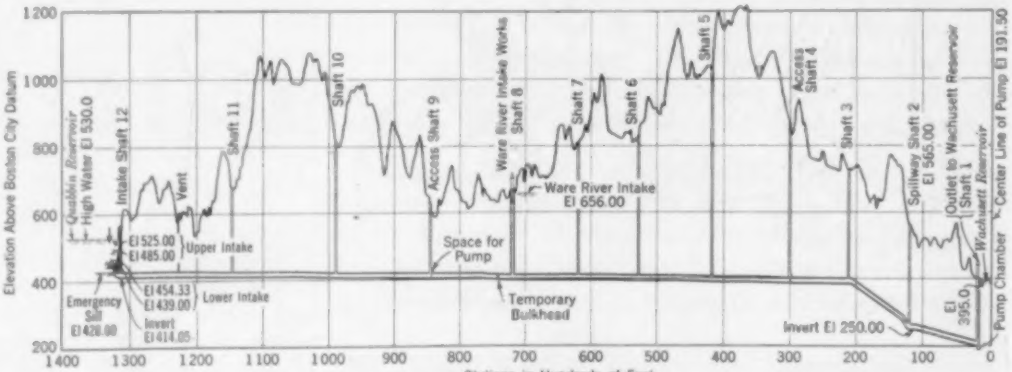


FIG. 2. PROFILE OF QUABBIN AQUEDUCT TUNNEL

only at times when the flow exceeds 85 mgd (131+ cu ft per sec), which is about the amount of the average flow. In other words, it is a diversion of flood waters. Therefore it is not economical to install the power plant until the Quabbin Reservoir is built. In 1931 the tunnel to the Wachusett Reservoir from the Ware River was completed, and diversions have been made as needed since that time. With but slight additional construction, this tunnel may also be used to divert water from the Quinapoxet River at Shaft 2. Such water may be routed either to the Wachusett Reservoir to maintain its level and for power, or if that basin is sufficiently full, to the Quabbin Reservoir for storage.

Considering the tributary watershed of 186 sq miles, the size of the Quabbin Reservoir is greater than would ordinarily be provided. However, it should be remembered that the Ware River at Shaft 8 drains 98 sq miles and that the watershed of the Quinapoxet River at Shaft 2 is 44 sq miles. Also, as seems probable, if other waters are needed in the distant future they may be stored in this reservoir. The capacity of the tunnel is sufficient to take from the Ware the maximum flow from a flood that is likely to occur but once in a hundred years.

In March 1928, the War Department made a ruling on the diversion of the Ware. At the end of that month a contract for 14 miles of the tunnel to Shaft 8 was awarded on bids previously received. Water from the Ware was first diverted through this section on March 21, 1931, immediately following the final decision of the U. S. Supreme Court as to the legality of the diversion. A contract for the extension of the tunnel about 11 miles to the Quabbin Reservoir was let in April 1931, and this work is nearing completion.

The shafts, 12 in number, vary in depth from about 100 to 656 ft. The capacity of the tunnel from the Quabbin to the Wachusett Reservoir is about 500 mgd when the former is at one-third its maximum height. From Shaft 8 eastward to the Wachusett Reservoir, about 1,200 mgd can flow through the tunnel and nearly an equal amount can flow westward into the Quabbin Reservoir, making a total of over 2,000 mgd (3,100 cu ft per sec) that can be diverted from the Ware when routed at the same time in both directions. It was not necessary to make the tunnel any larger than otherwise needed in order to carry the flood flow of the Ware, since the extra head at Shaft 8 will be sufficient to force the water through.

A cross section of the tunnel is shown in Fig. 3. The rock broke very close to the established lines. The cost of the work was materially less than the engineer's preliminary estimate, because the rock was sound and required very little temporary support, and a minimum thickness of concrete lining was generally used. Specifications require that on the side walls and invert no rock shall remain within the "A-line," which is fixed as a minimum at 3 in. from the face of the masonry on the side walls and invert and 8 in. in the arch. The contractor is paid for rock excavation and for concrete to a line fixed arbitrarily 12 in. outside the A-line on the side walls and arch and 2 in. outside this line on the invert. The average distance between the A and B lines on the side walls

Timbered Section

Before and After Lining

TUNNEL CONSTRUCTION, QUABBIN AQUEDUCT

and arch for the entire tunnel is $10^1/_2$ in., so that the actual amount of rock excavation was 2.6 per cent less, and that of concrete lining 8.4 per cent less, than the pay quantities.

In the whole 25 miles of tunnel, only 1,970 ft of timbering was required, or about $1^1/_2$ per cent of the length. The rock is mostly a compact schist with granitic intrusions. There was no heavy ground in the entire line, but there were some stretches where rock fell because of slacking of material in seams when exposed to the air. In such sections, aggregating some 3 miles in length, the rock in the roof was treated with three coats of gunite to a total thickness of about an inch. The masonry was not generally made thicker in this kind of rock, and by the use of gunite the extra expense of enlarging the excavation and placing timber supports was avoided.

The Ware River is now being diverted at Shaft 8 as required, to the Wachusett Reservoir. To prevent the water flowing westward to the Quabbin Reservoir, a heavy temporary bulkhead has been built about 2,200 ft west of Shaft 8 (Fig. 2) in the form of two large gates of steel, so designed as to withstand the entire head from the Ware River at its maximum, that is, about 265 ft. It is expected that this construction will be removed in six or seven years, when the Quabbin Reservoir is ready to receive water. At that time the full capacity of 2,000 mgd will divide at the bottom of the shaft, about half going to the Quabbin Reservoir and half to the Wachusett Reservoir.

The Ware River intake works are of special interest. The diversion dam, which is arched in plan and has a spillway crest 174 ft long at elevation 657, diverts the water into the shaft, the diversion being accomplished by nine siphon spillways. These siphons discharge into a central pit in the building, in which the water level is controlled by four butterfly valves, operated by oil pressure cylinders, which are set directly

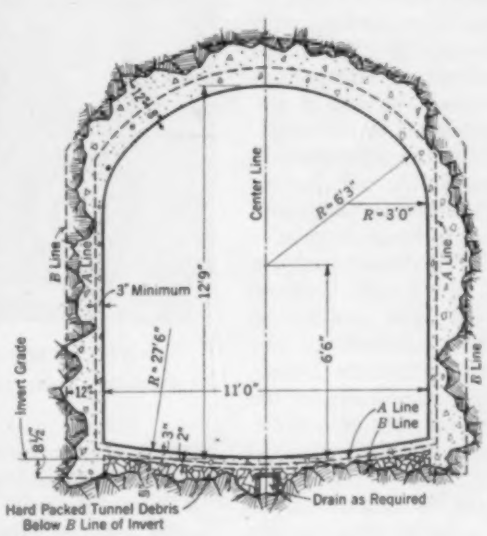
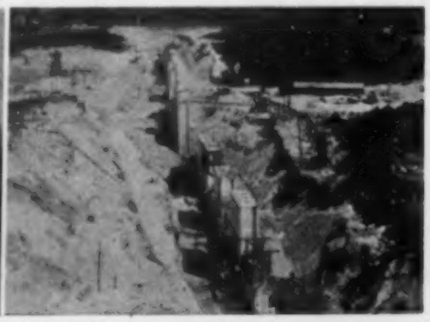


FIG. 3. TYPICAL TUNNEL SECTION

700 Ft of Circular Section, 12 Ft 9 In. in Diameter, Was Built Adjacent to Shafts 1 and 12

over the shaft in a floor or plug of reinforced concrete 9 ft thick. The valves throttle the discharge into the shaft, automatically maintaining the correct rate of diversion through the spillways by controlling the head on them and at the same time creating a water seal to prevent the entrance of air into the tunnel. For ordinary operation only a small head is required on the siphons.

A Caisson Seated Ready for Concrete



Construction Progress, August 1933

CONCRETE CAISSON WALL AT DIKE, QUABBIN RESERVOIR

Operation is entirely automatic, designed to divert all flood flows in excess of a rate of 85 mgd into the shaft, and to prevent any diversion when the flow in the river is less than this limiting rate. The nine siphon spillways, constructed in the foundation walls of the intake building between the river and the shaft, vary in capacity from 60 to 550 cu ft per sec under a 9-ft head. The crests of these siphons are 1 ft below the level of the main spillway. Siphon break pipes are set so that whenever the flow of the river is less than 85 mgd the siphons are automatically broken and no water can be diverted into the tunnel. On a rising flood the rapid priming of the siphons in succession, one after the other, up to the required capacity, is assisted by a series of float switches specially designed for sensitiveness and automatically connecting each siphon in its turn to a vacuum pipe system in the building, which is tapped into a 52-in. Venturi meter just below its throat, thus maintaining a vacuum in the entire system.

As the water is discharged from the four butterfly valves through a nozzle, it impinges tangentially, at an angle of about 30 deg with the horizontal, at high velocity against the cast-iron lining of the shaft. The helical vanes on this lining cause a rotation of the falling water, which is pressed by centrifugal force into a thin film against the cast-iron wall of the shaft. That the entrance of air into the tunnel is effectively prevented by the valve-controlled water cushion at the top of the shaft is shown by the high vacuum observed in the shaft.

Shaft 1, at the Wachusett end of the tunnel, where it is expected that a power plant eventually will be installed, is lined as two shafts, one a waterway shaft and the other a pump shaft. The latter is used whenever it is desired to unwater the three miles of tunnel that are below the hydraulic grade at all times. It is lined throughout with steel and with concrete inside the steel to prevent corrosion. The suction pipe of the pump extends through the steel and concrete lining. The pump discharges through a pipe embedded in the masonry and leading to the downstream side of a bulkhead that is placed in the gate house when the tunnel is being unwatered.

Eventually, when the power plant is installed, the waterway shaft will be covered by a large steel casting which will take the pressure from the Quabbin Reservoir. This shaft is anchored by a steel lining extending into the rock to a depth of 65 ft.

On the west end of the tunnel, the intake from the Quabbin Reservoir is about five miles above the main dam. It was located there for several reasons: one, to get proper rock support for the tunnel, and another, to make the main tunnel as short as possible. The water in this reservoir will rise about 98 ft above the intake. All but 10 billion gallons of the 415 billion gallons in the Quabbin Reservoir can be sent through the tunnel.

Two earth dams are required to complete the rim of the reservoir, one known as the main dam (Fig. 4), where the river now flows, and the other known for convenience as the dike (Fig. 5). Both these valleys are filled with a porous glacial deposit of sand and gravel to a maximum depth of about 130 ft. A tight cut-off wall to rock through this porous material at each location will be provided to minimize seepage under the future embankment. At the main dam the river diversion works have been built, and an exploratory caisson sunk, to determine the character of the glacial drift more definitely than could be done by borings and to demonstrate the possibility of lowering ground water by pumping.

In order to divert the river, a 30-ft tunnel 1,230 ft long was driven under the western hillside, with provision for plugging with concrete under an upstream gate house when the dam is completed. For the release of water through the dam, a 48-in. pipe has been laid in the tunnel masonry and a second pipe of the same size will be laid in the tunnel after it has been plugged. These pipes will discharge into the river the quantities prescribed by the Massachusetts law and by the War Department and will be available for other future use. At the dike, diversion works are unimportant because the drainage area is negligible—only $1^1/_2$ sq miles. The dike will contain $2^1/_2$ million yards of earth, as compared with 4 million for the main dam.

For a cut-off wall in the foundations of the dike, 31 reinforced concrete caissons in a line have been sunk

TUNNEL FOR DIVERSION OF RIVER AT MAIN DAM
At Upstream Gate House Where Masonry Plug Will Be Placed

to rock across the valley. These caissons, with closures, will form a continuous masonry diaphragm. The top 25 ft of this wall will be encased with impervious soil backfilled and rolled in the open trench cut. The backfill will have a thickness at the top of the wall of about 100 ft. It is expected that the dike embankment above the concrete cut-off wall will generally be built by a

hydraulic method. The first caisson, for exploratory purposes, was made 12 ft wide and 32 ft long. The remaining caissons in the dike are 45 ft long by 9 ft wide, with three wells, each 4 ft in diameter, the middle one being a man well and the two outer ones, material wells. The exploratory caisson was equipped with pumps, which demonstrated that the ground water could be lowered effectively, after which a contract for the remaining 30 caissons was let.

Had the ground water not been lowered by pumping from the caissons, many of them would have been subjected to a 130-ft head at the bottom of the excavation. The practicability of lowering ground water was proved so conclusively in advance that in general bidders as-

TABLE I. AIR PRESSURES IN CAISSON CORE WALL OF DIKE

Pressure In Lb per Sq In.	Aggregate Depth of Caissons, in Feet: With Ground Water as Lowered by Pumping (Actual Conditions)	With Ground Water Assumed to Be at Bottom of Trench
Free Air	1,454	0
0–10	419	679
10–18	163	523
18–24	18	338
24–28	0	202
28–33	0	210
33–38	0	91
38–41 max.	0	11
	2,054	2,054

sumed substantially reduced air pressures, and the cost of the work to the Water Supply Commission will be only about half the earlier estimates.

The caissons were built with dished ends 18 in. deep and were set 18 in. apart, thus leaving a maximum space of 54 in. between caissons. This space was finally excavated to rock and filled with concrete, thus completing a solid concrete wall across the valley.

A trench, which in the lower part of the rock valley was from 30 to 60 ft in depth, was first excavated, the ground water having been previously lowered at least to this elevation. In this trench the caisson shoes were set. Even if the ground water had been lowered only to the bottom of this trench, the air pressure required would have been greatly reduced, but since the actual

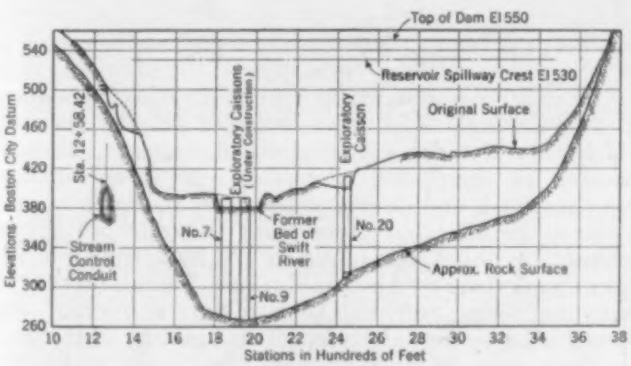


FIG. 4. CENTER-LINE PROFILE OF THE QUABBIN DAM

lowering was well below the bottom of the trench (Table I), a still greater reduction in air pressure was effected. With no preliminary pumping, the pressure at the bottom of 15 caissons would have exceeded 50 lb.

Table I does not include the pressures in the exploratory caisson, which were kept within 50 lb per sq in. by first sinking the caisson until the pressure reached 30 lb per sq in., and then lowering the ground water by pumping from its working chamber to permit the excavation in the dry of a nearby sheeted well, to a depth of 55 ft. The pumps were then shifted to this

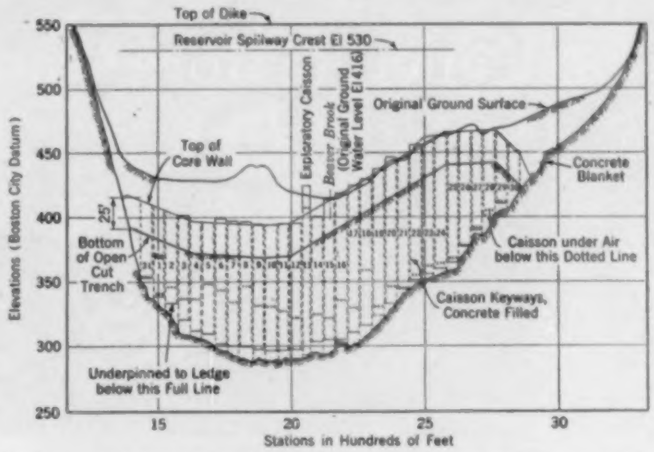


FIG. 5. CAISSON CORE WALL FOR THE QUABBIN RESERVOIR DIKE
Profile on Center Line

well and the caisson was sunk under a maximum pressure of 48 lb to rock before the pumps were again installed in it.

At the main dam, the first exploratory caisson could not be sunk in the lowest point of the rock valley on account of the proximity of the river and railroad and because work on the caisson would interfere with the construction of the diversion works. This caisson (No. 20, Fig. 4) was therefore located where the rock elevation is about 50 ft above that in the deepest part of the valley. While the lowering of ground water by pumping from this caisson has been satisfactorily accomplished, it has been decided to demonstrate the behavior of the ground water in the deeper part of the rock valley by pumping from two additional exploratory caissons, Nos. 7 and 9, now being sunk. The results of pumping from these caissons will be available before bids are invited on the cut-off wall. More pumping may be required for lowering ground water at the dam, but it appears reasonably certain that results as satisfactory as those secured at the dike can be obtained.

It was possible to divide the construction of the entire project into three steps, thus lessening the financial burden on the District. The first step was accomplished in March 1931, when the Wachusett Reservoir was connected by tunnel with the Ware River. It involved an expenditure of about $11,000,000. The second step consists in bringing additional water from the Swift River without storage, which can be accomplished in the spring of 1935. The last step, the completion of the Quabbin Reservoir, will probably be accomplished in the spring of 1940. It was calculated that the increased supply added in 1931 would last until 1935, and that the further increase in 1935 would last until 1940, when the third and final step would be completed. The legislation provides for a total expenditure for the Ware-Swift works of $65,000,000, of which about $27,000,000 had been expended up to March 1, 1934.

The work is being carried on by the Metropolitan District Water Supply Commission, of which Davis B. Keniston is Commissioner, and Charles M. Davenport and Thomas D. Lavelle are Associate Commissioners. Karl R. Kennison, M. Am. Soc. C.E., is Designing Engineer and Principal Assistant of the Chief Engineer. W. W. Peabody, N. LeRoy Hammond, Members Am. Soc. C.E., and R. R. Bradbury are Division Engineers.

Concrete for Madden Dam

A Strict Yet Flexible Laboratory Control, Based on Water-Cement Ratio and Trial Method, Results in a Product of High Quality

By Irwin E. Burks

Chief Concrete Technician, Madden Dam Division, The Panama Canal, Canal Zone

Madden Dam on April 9, 1934
Mixing Plant Is Just Out of the Picture, on the Right

That a radical change has taken place in methods of concrete making is illustrated by the swing from the extremely dry mixes used in the early years of the century to the very wet ones common from 1910 to 1915, and finally back toward the relatively stiff mixes employed today. Although authorities may still disagree on many points, one fundamental principle seems to have been established and almost universally accepted—that extremely wet mixes do not result in a durable concrete. In this article, prepared from a paper recently read before the Panama Section of the Society, Mr. Burks outlines the principal features of the concrete specifications for the dam, describes the preliminary tests of local aggregate made, and summarizes the procedure adopted to secure a precise laboratory control of field mixing and placing.

Formulation of proper rules for the proportioning of aggregates for concrete, the scientific design of mixes, and the testing of materials, has been the aim of active investigation and study from 1900 to the present time. Although these questions received the serious attention of engineers and constructors from twenty to thirty years ago, probably 95 per cent of the concrete placed prior to 1915 was mixed by the old familiar method of arbitrary proportions by volume. Perhaps this method may be said to have one point in its favor—simplicity. It has been shown that concrete proportioned in this way will have neither uniform strength nor constant yield because of the influence of the grading of aggregates on these factors.

About 1905 the late William B. Fuller, M. Am. Soc. C.E., and Sanford E. Thompson, M. Am. Soc. C.E., recognizing the inadequacy of the method of arbitrary proportioning by volume, made some studies designed to show the effect of different gradations of aggregate on the strength and density of concrete. An account of them was published in Transactions, Vol. 59, under the title, "The Laws of Proportioning Concrete." An "ideal gradation curve" was developed, which was intended to show the correct proportions of fine and coarse aggregate. In 1922, when I attempted to use the Fuller and Thompson "ideal curve" on a paving project in New Mexico, it was found that even the normal fluctuations in pit gravel would upset the calculations and that the mixes resulting from this method were usually harsh and unworkable.

During 1915 the Portland Cement Association established a laboratory at the Lewis Institute, Chicago, for the purpose of studying problems of concrete making, with Duff A. Abrams, M. Am. Soc. C.E., in charge of the work. The first report was published in 1918 as Bulletin No. 1 of the Lewis Institute, and was entitled *The Design of Concrete Mixtures*. The publication of this bulletin marks the introduction of the theory that there is a fundamental relation between the strength of the concrete and the ratio of water to cement in the mix—a relation which plays an important part in our present-day methods of design. The law of the water-cement ratio may be stated as follows:

> For plastic mixtures, using sound aggregates, the strength and other properties of concrete under given job conditions are controlled by the net quantity of water used per sack of cement.

As a means of applying the principle of the water-cement ratio, in 1918 Abrams offered the fineness-modulus method of designing concrete mixtures. The fineness modulus is a function of the aggregate, which is found by dividing by 100 the sum of the percentages in the sieve analysis retained on a set of sieves with openings of specified sizes. He gave a rather lengthy formula which uses the desired strength of concrete, fineness modulus of aggregates, weight of aggregates separate and combined, and desired consistency of concrete, and is solved for the field proportions either by weight or volume. Without detracting in the least from the excellent work done by Abrams in presenting this method of designing mixes, and with no attempt to question the theoretical exactness of his formula, it may be stated that the fineness-modulus method was never used extensively in practice and was not popular with the majority of men in the field. It should be remembered that when this method was first put on trial with construction men, the process of aggregate classification and cleaning had not reached its present high state of development. It was simply a case of the laboratory being too far in advance of the field. As one contractor expressed it after a serious attempt to use the new method, "I spent more money for testing sieves and pencils than for mixer blades."

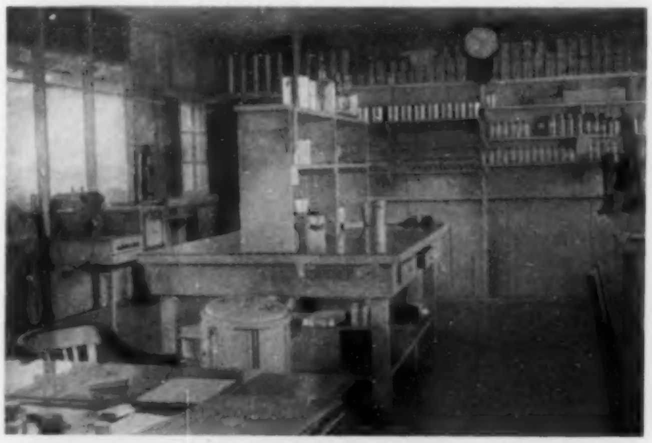
Regardless of the difficulties encountered in the practical application of the fineness-modulus method, the

value of the principle established by Abrams was not lost on engineers, who were searching for a new and better way of designing concrete mixtures. Numerous cases are on record in which the fineness modulus was used in the preliminary design of mixtures, and if the resulting concrete was not of the desired consistency and did not possess the proper degree of workability, corrections were made by arbitrarily adjusting the quantities of fine and coarse aggregate, but without altering the quantities of cement and water. It will be seen that this procedure preserved the fundamental principle of Abrams' law of the water-cement ratio. The use of this modified fineness-modulus method grew in popularity from 1920 to 1926 and really marked the birth of our present-day "trial method" of mixture design, although it was not officially known by that name until 1926, when F. R. McMillan, M. Am. Soc. C.E., published a description of the method in the Portland Cement Association's bulletin, *The Design of Concrete Mixtures* (1926).

As a result of my own experience I believe that the water-cement ratio provides the most satisfactory basis for designing concrete mixtures yet proposed. Its simplicity and ease of application are refreshing to the man in the field, coming as it does after a lengthy period of struggle and uncertainty with some of the more intricate and complex theories, which, although easily understood and applied in the laboratory, are quite impractical in the field.

SPECIFICATIONS FOR MADDEN DAM CONCRETE

The specifications for the Madden Dam and appurtenant works in the Canal Zone provide that cement for the concrete shall be furnished by the Government and



INTERIOR VIEW OF CONCRETE TESTING LABORATORY

that the aggregates shall be obtained from Government-owned natural deposits. The usual requirements for quality of aggregates are included, and the matter of grading is covered by the statement, "It [the gravel] shall be so graded that concrete of the desired workability, density, and strength can be made without the use of excess sand, water, or cement." It is also stipulated that, in so far as practicable, the aggregate used shall consist of the entire yield of natural deposits. The sand is required to pass a No. 4 standard sieve, and the fineness modulus is limited to a minimum of 2.75 and a maximum of 3.25. The gravel is divided into the four sizes given in Table I.

TABLE I. CLASSIFICATION OF GRAVEL

GRAVEL NO.	SIZE (ROUND OPENINGS)
1	4 mesh to 7/8 in.
2	7/8 in. to 1 3/4 in.
3	1 3/4 in. to 3 in.
4	3 in. to 8 in.

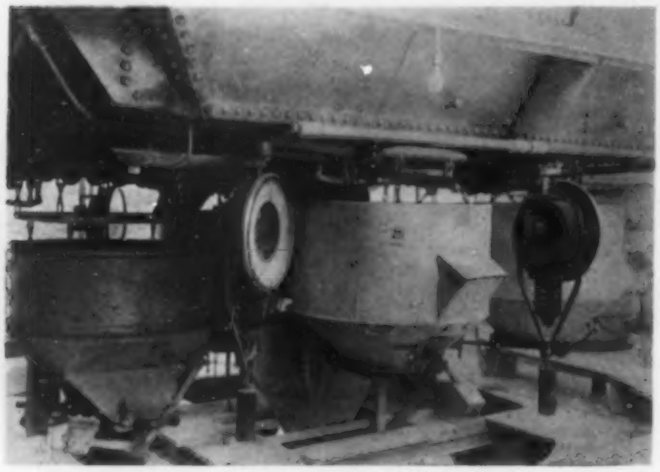
No definite method for proportioning the materials is stated in the specifications. The minimum compressive strength for mass concrete in the dam is specified at 1,500 lb per sq in. at the age of 28 days, and a minimum of 3,500 lb per sq in. is specified for beams, slabs, and other reinforced members. Presumably these strength requirements were based on tests of standard 6 by 12-in. cylindrical specimens, because a clause has been inserted which provides that where the size of the test cylinder or character of the concrete used is such that the cylinder strength is not directly indicative of the strength of the concrete entering the work, appropriate conversion or equalizing factors shall be applied to the cylinder strength obtained. The only reference to the matter of proportioning is a statement that the mixes shall be such as will result in concrete having suitable workability, density, impermeability, and strength without the use of an excessive amount of cement.

AGGREGATE CLASSIFICATION PLANT AND CENTRAL MIXING PLANT
Testing Laboratory Indicated by a Cross

While the specifications require accurate control of the quantity of mixing water and provide for adjustments to allow for variations in the moisture content of the aggregates, it is stated that the amount of water may be changed to secure concrete of proper consistency. This statement appears to be somewhat inconsistent and does not harmonize exactly with the principles of the water-cement ratio law. However, it was assumed that it was not the intention to permit any very great change in the quantity of mixing water to improve consistency, since such changes would affect the strength of the concrete. In actual practice the permissible variation in water content is set within a specified range, beyond which corrections in consistency are made by variations in the quantities of aggregate rather than by further changes in the amount of mixing water. The specifications recognize the value of plastic consistencies and prohibit the use of over-wet mixtures by limiting the slump to 3 in. for mass concrete and 6 in. for thin reinforced walls or slabs, or for positions where placing is difficult.

Although the specifications define the strength range of the concrete to be used on the project as from 1,500 to 3,500 lb per sq in., it is recognized that a more definite classification of the concrete according to strength and other characteristics is essential and provides a convenient and understandable means of designating the quality desired for different sections of the work. Therefore, the concrete has been divided into the four strength classes given in Table II.

The strengths classified as A, B, C, and D may be produced by using the smallest size, or a combination of all four sizes of coarse aggregate available on the project, as listed in Table I. Further to identify the mixture, the letter symbol giving the strength class of the concrete is followed by a number symbol indicating the maximum size of gravel used. Thus "A-4" would indicate Class A concrete in which all sizes of gravel up to 8-in. were used.



GRAVEL BATCHER AND SCALES, CENTRAL MIXING PLANT
Cement Batcher at Low Center

In any combination, the proportions of the different sizes of gravel may be manipulated within a certain range without affecting the strength of the concrete and by only slightly influencing the cement factor. The object of manipulating the different sizes is to conform as nearly as practicable with the grading of the run-of-the-pit gravel and to avoid waste of material. Laboratory investigations and tests made prior to the beginning of concrete placing showed that the grading of gravel would fall within the ranges indicated in Table III.

TABLE II. CLASSIFICATION OF CONCRETE ACCORDING TO STRENGTH

Class	Design Strength at 28 Days Lb per Sq In.	Water-Cement Ratio By Volume	Cement Factor Bbl per Cu Yd
A	1,750 to 2,250	1.05	1.00
B	2,250 to 2,750	0.95	1.10 to 1.20
C	2,750 to 3,250	0.85	1.25 to 1.35
D	3,250 to 3,750	0.75	1.40 to 1.50

A letter which forms the third symbol of the mixture identification indicates the various combinations. Thus "A-4-B" indicates Class A concrete with a maximum size of coarse aggregate of 8 in., combined as follows: 32 per cent of size No. 1; 28 per cent of size No. 2; 20 per cent of size No. 3; and 20 per cent of size No. 4 (Tables II and III). The water-cement ratios given in Table II are maintained as nearly as practicable, and the cement factors shown cover concrete of average consistency ($1^1/_2$ to $3^1/_2$-in. slump). Obviously, an increase in slump greater than the range indicated would be reflected by an increase in the cement factor, assuming that the water-cement ratio remained constant.

A study of the data in Tables I, II, and III will show that each strength class of concrete has four basic mixes: that is, each class may be produced by using No. 1, No. 2, No. 3, or No. 4 gravel as the maximum size. This results in a total of 16 different aggregate combinations, any one of which might be required to meet specified strengths or different placing conditions on the project. With these data as a starting point, the concrete inspection force assumed as its first duty the design of these 16 mixtures. The work was undertaken as soon as the necessary laboratory equipment could be assembled and was finished before the contractor's mixing plant was put in operation.

TABLE III. PERMISSIBLE PROPORTIONS OF GRAVEL SIZES

Size Symbol	Percentage of Each Size: No. 1	No. 2	No. 3	No. 4	Fineness Modulus
2-A	15	85	0	0	6.85
2-B	25	75	0	0	7.00
2-C	35	65	0	0	7.15
2-D	50	50	0	0	7.35
3-A	42	48	10	0	7.55
3-B	37	43	20	0	7.71
3-C	32	38	30	0	7.88
3-D	27	33	40	0	8.13
4-A	37	33	20	10	7.89
4-B	32	28	20	20	8.16
4-C	27	23	20	30	8.43
4-D	22	18	20	40	8.70
Average run of the pit	35	30	23	12	8.00

PRELIMINARY TESTS

Preliminary work consisted of sinking test pits in the gravel bars in the area selected by the contractor for the reclaiming of his aggregate. The pits were located so as to be representative of the entire area and were carried to the same depth as that to which the contractor intended to work. This investigation involved the actual hand-screening of about 30 tons of raw material. The grading of the material from the individual pits was analyzed, and the average for the entire area was studied. The sodium sulfate test was used to indicate the soundness of the gravel, and tests of tensile strength were made on the sand. The results of these tests showed the quality of the material to be equal to or above specification requirements. Part of the material removed from the pits was washed, separated into the sizes designated by the specifications, and delivered to the laboratory to be used in studying the proportions of the mixes.

CONCRETE TRAIN AND 8-CU YD BUCKET

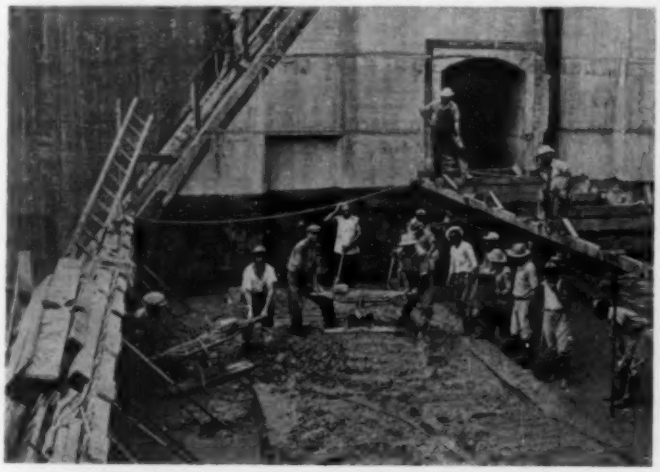
In designing these mixes at the laboratory, the fineness modulus method was used to arrive at the approximate proportions of cement and fine and coarse aggregate. Then the water-cement ratio for the desired strength was selected from Abrams' curves, and the various ingredients were put together in the calculated amounts and mixed in a half-bag mixer. The resulting concrete was dumped on a platform and its workability and texture judged by the ease with which it could be puddled

and molded into the test specimens. Usually two or three trials were sufficient to correct deficiencies in workability. The mixes based on fineness modulus generally were harsh and difficult to work. When this was the case, the amount of coarse aggregate was reduced and that of fine aggregate increased a like amount. The extent of the aggregate adjustment was purely arbitrary, and the approach to ideal workability was made in "trial" steps but was accomplished without changes in the amount of water and cement.

Once the proper proportions had been worked out by trial, the yield, cement factor, and density were calculated by the method of absolute volume. The figures were then checked by actual measurements. To avoid errors, check batches were mixed on separate days. Between 30 and 40 specimens for compression tests were made for each class of concrete, and the test results were plotted for comparison with Abrams' strength curves. At the end of the 28-day period the strength curve developed from the laboratory specimens was found to be parallel to, and 15 per cent above Abrams' curve. The strengths at the ages of 3 and 7 days were found to be 39 and 58 per cent, respectively, of the 28-day strength. The laboratory technique used in making the tests followed standard practice.

A study of all the factors developed through the tests and experiments made with the 16 different combinations of aggregates seemed to justify the following conclusions:

1. The compressive strength of concrete is not greatly affected by a rather wide variation in the gradation of the coarse aggregate as long as the water-cement ratio remains constant and the mix is workable.
2. For identical water-cement ratios, the cement factor is greater for unbalanced grading of coarse aggregate than for uniform grading.



Concrete Being Tamped with an Electric Vibrator
Man on Scaffolding at Right Is in Telephonic Communication with Mixing Plant

3. For mixes of average consistency (with a slump of from 3 to 4 in.) and of identical water-cement ratios, the cement factor decreases as the maximum size of the coarse aggregate increases.
4. The average strengths of the 16 mixtures were all above the lower limits of the design range; therefore the proportions worked out in the laboratory experiments were considered satisfactory for use on the project.

CONTROL OF MIXING

When the basic mixtures had been thus developed, attention was turned to the mechanics of using them in the contractor's central mixing plant, where all proportioning is done by weight, using standard scales. In approaching the problem of mixture control at the central plant, the objective was to develop a system that would, first, enable the Government inspectors accurately to apply the laboratory data to all conceivable conditions of operation, such as changes in the moisture content of

Progress of Construction on June 30, 1933

aggregates, necessary changes in consistency, and variations in size of batch; and second, to enable this to be done with the least possible delay to the contractor's work.

If the aggregates were dry, or if they contained a uniform amount of moisture, the problem of mix control would amount to little more than setting the batcher scales for the desired class of concrete and allowing the mixers to grind it out indefinitely. However, this is not the case. At Madden Dam the lack of sufficient storage capacity for washed and classified aggregates has resulted in troublesome fluctuations in the moisture content of the sand and gravel. This necessitates frequent adjustments in batch weights. A change of 1 per cent in the moisture content of the sand causes a noticeable change in the consistency of the concrete. The correction involves a change in the weight of the batches of sand and water.

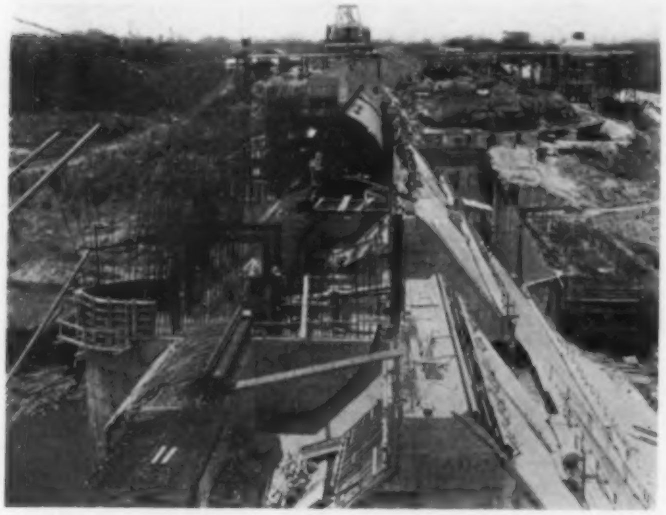
WIDE RANGE OF MIXES AVAILABLE

In anticipation of these fluctuations, and to enable the inspectors to make the necessary corrections speedily, without resort to paper and pencil, a very complete set of tables has been prepared which cover any moisture conditions from zero to 10 per cent. In these tables the proportions of the mix are based on 100 lb of cement. This feature enables the inspector to adjust his scales promptly for any size of batch from about 5 cu ft to the capacity of the mixers, and for any moisture content within the range stated. The importance of this detail appears greater if it is considered that there are 320 possible combinations of scale settings due to moisture changes alone for the four classes of concrete used.

Another factor, which may also be charged against insufficient storage space, is the occasional shortage of one or more of the four sizes of gravel. When this occurs the inspector has two possible courses: either he can adjust his gravel combination or he can discontinue production of concrete until the supply of the deficient sizes has been replenished. Since it had been recognized that a rather wide variation of gravel grading had very

little effect on the strength of the concrete, it appeared that the logical policy would be to adjust the grading of the gravel within the permissible range shown in Table III rather than to delay the work.

On the batcher floor of the plant is located the office of the Government's inspector in charge of mixing operations. There he has a clear view of all the scale dials and can observe the accuracy with which the operators charge the batchers. The office is provided with the necessary



INSTALLING DRUM GATES IN THE SPILLWAY SECTION
Elevated Tracks from Mixing Plant to Placing Cableway in Background

apparatus for making moisture tests and has a telephone which connects with the inspector in charge of placing operations at the forms and also with the laboratory. By a system of bells and signal lights, he is able to communicate with other sections of the plant and to know what is going on without leaving his post. In addition to the oral instructions given during the period of initial training, each inspector is provided with a manual of instructions which contains all mixture design tables, aggregate grading charts, specifications as to the class of concrete to be used in all sections of the work, and other information necessary to the performance of his duties.

The regular control procedure during any period of plant operation is substantially as follows. When reporting for duty, the inspector is notified of the section of the structure for which concrete is to be mixed. His instruction manual indicates the class of concrete to be used. He makes a test to determine the moisture in the aggregates, and from his design tables he selects the proper proportions of the mix. The points on the scale dials are set accordingly, and the plant crew stand by for instructions from the forms to start mixing. The inspector and foreman in charge of placing operations agree on a rate of delivery estimated to fit the conditions at hand. This information is telephoned to the plant inspector, who then regulates the speed of production.

Consistency of concrete is regulated by slump tests made at the point of placement, whence the results are telephoned to the plant inspector. As a further check on consistency, the inspector may require the laboratory to make slump tests at the plant. Tests are made at regular intervals during a shift to show the moisture in aggregates. This is done regardless of apparent uniform consistency of concrete. At the end of each shift the inspectors are required to submit a detailed report showing the work done, the classes and amount of concrete produced, and the quantity of materials used.

As a check on the grading and cleanness of the gravel, one sample representing each size is tested each 24 hr, and at least three samples of sand are tested during the same period. If the test results indicate deficiencies in the properties mentioned, the contractor is notified and corrections are made at the plant. A sample is taken to represent each class of concrete produced during an 8-hr shift. The majority of these are obtained at the central mixing plant, although check samples are taken periodically at the forms. A sample consists of enough concrete to make at least six 6 by 12-in. cylinders, which are broken at various ages from three days to three years.

Before the cylinders are formed, the samples of concrete are passed through a screen having openings 2 in. square, and the coarse aggregate that will not pass this screen is discarded. The effect of this procedure on the compressive strength of the concrete has been under investigation for a number of years. Perhaps the most comprehensive program of investigation ever undertaken to develop a ratio between the strength of standard laboratory specimens and the strength of the concrete in the structure itself was that conducted by the U. S. Bureau of Reclamation in connection with the building of Boulder Dam. Progress reports indicated that the small laboratory specimens gave strengths that were from 12 to 25 per cent higher than those of the unaltered concrete when tested in cylinders 3 ft in diameter and 6 ft high.

TABLE IV. RESULTS OF COMPRESSIVE TESTS ON 6 BY 12-IN. CYLINDERS

AGE	CLASSES OF CONCRETE Strength in Pounds per Square Inch			
	A	B	C	D
3 days	1,236	929	1,237	1,980*
7 days	1,837	1,952	2,266	2,962†
28 days	2,885	3,095	3,550	4,110
3 months	3,457	3,910	4,160	4,289
6 months	3,800	4,576	4,186	.....
9 months	3,945	4,423	4,715	.....
1 year	3,980	4,560*	4,847	.....

* One sample only. † Two samples only.

From the beginning of concrete placing to April 1, 1934, about 3,150 specimens of concrete were tested. The results of these tests are summarized in Table IV. A correction factor of about 20 per cent should be applied to the strengths shown for Class A concrete, and one of 18 per cent for those of Class B concrete. This is to compensate for the removal of the larger sizes of coarse aggregate from the samples. No correction is necessary for concrete of Classes C and D because the maximum size of gravel used in these classes made it possible to test the mixtures without alteration.

METHOD PROVED SATISFACTORY

Tests show that the strength of the concrete in the Madden Dam complies with Government specifications and that it is slightly above the designed strength. The procedure followed—using the water-cement ratio for the mortar and the trial method for proportioning the coarse aggregate—has proved to be simple and workable for controlling the quality of the concrete and is recognized as a sound basis for the design of mixes.

Appreciation and thanks are here expressed to Col. C. S. Ridley, Engineer of Maintenance, the Panama Canal, and to E. S. Randolph, M. Am. Soc. C.E., Construction Engineer, Madden Dam Division, for their advice and cooperation in the work described in this article. Their active interest and support have been of great assistance.

Los Angeles Incinerates Sewage Screenings

Equipment Recently Installed Destroys 50 Tons of Wet Screenings Daily

By H. G. Smith

Engineer of Sewer Design, City of Los Angeles

In order to provide for the disposal of about 30 tons of sewage screenings daily, the engineers of the City of Los Angeles in 1933 entertained bids for furnishing and erecting an incinerator at the City's Hyperion screening plant. Previously the screenings had been buried in the sand dunes adjacent to the plant, but this method of disposal had to be abandoned because of scarcity of available land. In the accepted installation, a small and compact unit, the wet screenings are dewatered in a roll press to a moisture content of 70 per cent and burned in an oil-fired destructor at a temperature of about 1,600 F. After a rigid 90-day trial of the plant, in which its performance exceeded the guarantees both in capacity and in economy, the incinerator was accepted by the city and placed in regular operation in January 1934. In this article Mr. Smith outlines the specification requirements, the operating guarantees, and the performance record, under test, of this installation.

APPROXIMATELY 90 per cent of the sewage of the City of Los Angeles, amounting to about 130 mgd, flows through the Hyperion screening plant, the effluent being disposed of in the Pacific Ocean by means of an ocean outfall with the point of discharge 5,000 ft offshore.

Screening Plant at Hyperion
New Incinerator with Stack in Foreground; Large Buildings House Screens

Prior to entering the screen pits, the sewage passes through coarse grids. These are made up of vertical bars having a clearance of 1 in. and are continuously cleaned by automatic rakes. The screens, ten in number, with a capacity of 40 cu ft per sec each, are of the revolving cylindrical type, and the plates are perforated with slots $^1/_{16}$ in. wide and 2 in. long. From 3 to 6 per cent of the suspended solids in the sewage are removed by the screens and pass to pits immediately adjoining, from which they are recovered by bucket elevators dumping into cylindrical pneumatic ejectors.

Various methods for more satisfactory disposal of screenings have been under consideration for some years, but the means employed from the erection of the screening plant until recent months has been to convey them by pneumatic pipe line to the high sand dunes in the vicinity, where they have been buried in trenches previously excavated with drag-line equipment. Since these screenings, which accumulate at an average rate of 33 tons per day, are made up of small rags, matches, packing-house wastes, and seeds and peelings of seasonal fruit, with a small amount of fecal matter, decomposition is very slow. Therefore the method has proved unsatisfactory, as the burial areas have increased to considerable size with the passage of time.

INCINERATION SUBSTITUTED FOR BURIAL

Early in 1932, the final decision was reached to replace this method with some form of incineration. Accordingly location plans and specifications were at once prepared. Even though incineration of sewage screenings was a comparatively recent development, the specifications were very rigid as regards guarantees. However, they were broad enough to include practically all equipment on the market.

In essence, the specifications called for the furnishing and installing of suitable equipment, complete and ready for continuous operation, for the disposal of wet sewage screenings by incineration at a rate of 50 tons per 24 hr. The contractor was required to furnish all the concrete foundations, floors, and appurtenances necessary to complete the plant, with the exception of a building.

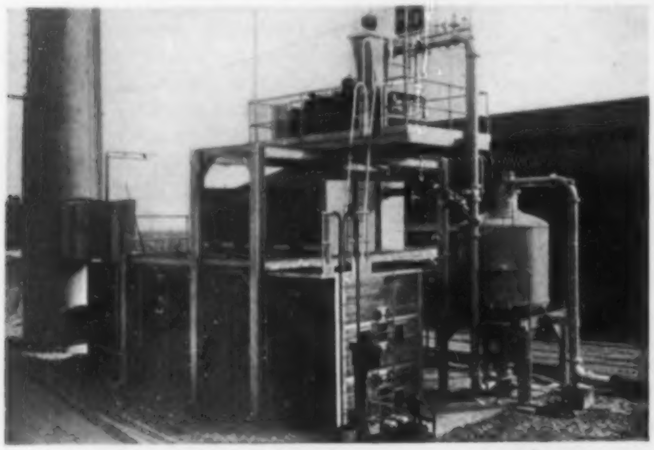
Each bidder was required to submit a list of all plants built by him for the incineration of the same or similar material; to state and guarantee the number of man-hours of labor needed, both common and skilled; to give the consumption of electric current, fuel oil, and fresh water of the plant; and to name other sources of energy needed or other supplies required to reduce the screenings to an ash containing not more than 1 per cent of organic matter. The specified rating of the plant was 50 tons of screenings having 85 per cent of moisture, 9 per cent of ash, and a heat energy of 8,000 Btu per dry pound. It was also specified that bidders should indicate and guarantee how much a change of 5 per cent in the given moisture content, a change of 500 Btu in heat energy, and one of ±5 per cent in ash content would affect the rated capacity and operating cost of the plant.

As regards operation of the furnace, the contractor was required to state the maximum and minimum temperatures necessary for maximum efficiency and to guarantee that practically no dust or sparks would issue from the chimney; that the smoke density, except when the fire was started in a cold furnace, would not be greater than that indicated by Chart No. 1 of Ringelman's smoke scale; and that at no time would the plant create a nuisance or emit offensive or obnoxious odors, gases, dust, or smoke.

The specifications set forth the requirements relative to such details as arrangement, plumbing and drainage, fresh-water piping, electrical wiring and lighting, grading, foundations, operating floor and construction, storage of screenings, stand-by equipment, tools and recording instruments, stairs, platforms, safety devices, and painting. On the other hand, the bidder was requested to state sizes and types of a proposed building to house the incinerator, of the moisture-reducing equipment and incinerator units, of the drying hearth and grates, ash pit and ash-disposal equipment, forced-draft and pre-heater combustion chamber, flue and stack, fire- and red-brick work, and fuel and oil-burning equipment.

Specifications required that on completion of the installation, the contractor was to make complete tests.

First, a tuning-up period of 4 or 5 days was permitted to synchronize the operation of the equipment. Then, commencing on the day following the end of the tuning-up period, the official test was to be made, lasting for three consecutive days of 24 hr each, for the purpose



SEWAGE SCREENINGS INCINERATOR READY FOR TEST
Capacity, 50 Tons of Wet Screenings per 24 Hr

of rating the plant with regard to all specified guarantees and the requirements of the specifications. During the tests the plant was to be operated at a rate of 50 tons of wet screenings per 24 hr.

In case this test failed to meet the guarantees, a second tuning-up period and official test was to be permitted, but not over 30 days were allowed to elapse between the beginning of the first official test and the completion of the final official test. In the event of failure to meet requirements within this specified time, the work was to be stopped and the plant rejected. Provided the official test was satisfactory, a trial test period was to follow and continue for 90 consecutive days for the purpose of testing the durability and maintenance requirements of the plant relative to material, construction, and workmanship. In the event of failure of the equipment to pass these tests satisfactorily, the contractor was to be allowed 30 days to remove the plant from the property.

On the basis of these specifications, the Board of Public Works received bids from several different incinerator manufacturers. After a detailed analysis of their bids, the different types of equipment proposed, and the guarantees offered, it was decided that the proposal of the Morse-Boulger Destructor Company best fitted the conditions and general situation. To this firm a contract was therefore awarded for the amount named in the bid, $32,500.

Briefly, this company proposed to erect a plant consisting of one destructor with a capacity of 25 tons of dewatered screenings per 24 hr, a 150-cu ft ejector with accessories, one roll press for dewatering screenings to a moisture content of 70 per cent, one bin with a capacity of 8 tons of dewatered screenings, a flue connection and chimney, a panel board for indicating and recording meters, and a concrete floor and foundations for all this equipment.

The design of the destructor is based on an average temperature of 1,300 F and a minimum of 1,200 F in the combustion chamber under normal operating conditions, and from 1,400 to 1,800 F in the main burning chamber.

In addition to meeting the general requirements of the specifications, the guarantees listed in Table I were made by the successful bidder.

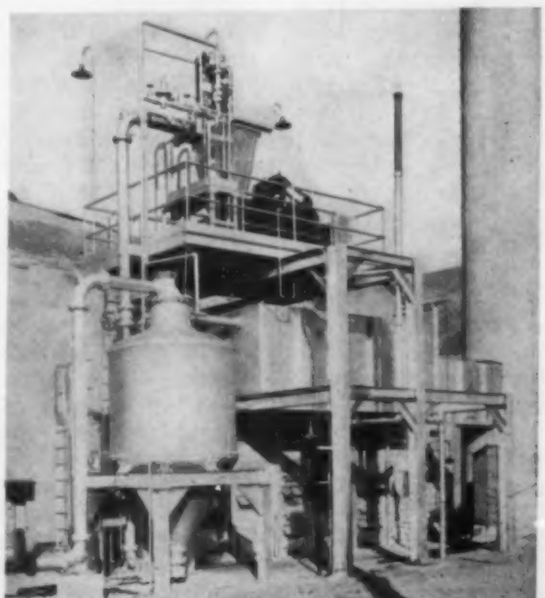
TABLE I. OPERATING GUARANTEES FOR VARIOUS CHARACTERISTICS OF SCREENINGS
Daily Requirements for Continuous Operation

ITEM	PERCENTAGE OF MOISTURE			BTU PER DRY CUBIC FOOT		PERCENTAGE OF ASH	
	80	85	90	7,500	8,500	4	14
Labor for stoking, in man-hours	24	24	24	24	24	24	24
Labor for charging and maintaining equipment, in man-hours	24	24	24	24	24	24	24
Fuel oil, in gallons*	655	490	360	563	417	480	500
Air, in cubic feet at 60 lb per sq in.	38,280	29,200	21,500	33,160	25,240	28,660	29,740
Electric energy, in kilowatt-hours	44	44	44	44	44	44	44
Fresh water	None	None	None	None	None	None	None

* Oil, based on 147,000 Btu per gal.

As constructed, the plant receives the screenings through a pneumatic pipe line leading into an ejector tank with a capacity of 200 cu ft, or 6 wet tons, which provides sufficient storage to handle a peak flow. This ejector is provided with alarms indicating when it is full and when empty. An air pressure of 15 lb is constantly maintained above the surface of the screenings. Under this pressure, which can be immediately raised in the event of stoppage, the screenings flow by hand-valve control into the hopper of a 24 by 8-in. continuous filtering machine operated by a 3-hp geared-head motor running at 675 rpm. This roll press has a guaranteed capacity of 3 tons per hour. The hopper of the press is also provided with light signals indicating when full and when empty.

The dewatered screenings passing from the rolls fall into an electrically operated scale box, which on receiving 100 lb automatically dumps its contents into the storage bin and records the operation on a counter. The water coming off the press is collected in a pan below the rolls and flows through a 2-in. pipe to a meter made up of a balanced double-compartment tank, each compartment of which automatically dumps when full and at the same time operates a counter. These weighing and metering devices were designed and installed by the city.



INCINERATOR EQUIPMENT BEFORE ENCLOSURE
Ejector Tank at Left, Continuous Filtering Machine on Top, and Storage Tank for Dewatered Screenings Immediately Over the Furnace

Should the press be temporarily out of order for any reason, a by-pass permits the wet screenings to be discharged directly from the ejector into the bin. The open storage bin is located immediately over the destructor, convenient to the charging holes. It is made of steel and has a capacity of 8 tons of pressed screenings. From the bin the screenings are raked by hand into the destructor through four charging holes, each 18 in. in diameter and provided with a pivoted sliding cover consisting of a 4-in. fire-clay slab set in a cast-iron frame and operated by a heavy bent handle.

BUILDING ENCLOSING INCINERATOR EQUIPMENT
By January 1934, Tests Were Completed and Equipment Was Placed in Operation

In exterior dimensions the destructor proper is 27 ft 9 in. in length, 8 ft 0 in. in width, and 9 ft 2 in. in height. The brickwork is protected and supported by heavy structural steel framing with six pairs of buckstays consisting of 6-in. I-beams. The walls are 24 in. in thickness and the fire-brick lining is insulated with diatomaceous brick 3 in. thick.

There are two main burning hearths with a total area of 96 sq ft, made up of fire-brick arches sprung across the destructor from skewbacks. The screenings raked into the charging holes fall directly on the upper main hearth and at once receive the direct heat from the primary oil burner. This hearth is provided with four 3-in. slots through which the burning screenings are gradually stoked to the lower main hearth. Meanwhile the burning gases pass forward over the upper hearth, downward and back over the lower hearth, then again down at the back end of the lower hearth into the combustion chamber (which has a volume of 258 cu ft), thence forward through the combustion chamber, up and through the flue, and finally into the chimney. From the primary oil burners to the entrance of the flue section, the gas travels a distance of 52 ft, measured along the center of the passages.

Two pyrometers connected to continuous recording devices have been installed, one at the point where the gas passes from the upper to the lower hearth and the other at the flue end of the combustion chamber. These usually indicate a temperature of 1,600 and 1,200 F, respectively. The ash is finally removed from the lower hearth directly into a hand truck. The flue connection has an area of 1,298 sq in. and is provided with a fire-clay damper bound in a frame of high-temperature steel. The chimney is of reinforced concrete, is 85 ft in height above the foundations, and has an inside diameter top and bottom of 36 in. The lower 35 ft is lined with fire brick 6 in. thick, and the upper 50 ft with the same material $4^1/_2$ in. thick.

The destructor is equipped with two oil burners of the high-pressure, external, atomizing type, of 20 gal per hr capacity. One burner is located in the main fire box over the upper hearth, the heat coming in direct contact with the freshly charged screenings. The other is situated at the end of the lower pass and is used as a secondary burner when starting up or when additional heat is needed in destroying exceptionally odorous gases. The burners are fed by two rotary fuel-oil pumps driven by $^1/_2$-hp motors geared to operate at 600 rpm. Meters are provided for the measurement of oil, electric current, and air, and strip-chart records are made continuously from the indicating pyrometers.

On the day following the tuning-up period, the official test commenced and continued without interruption for the required continuous period of 72 hr. During this time the plant was under constant observation by engineers representing the city. About three thousand individual entries and computations were made, from which eight tables were compiled in making up the report.

It developed that the actual average moisture content of the raw screenings was 83.5 per cent; the Btu value, 9,934; and the ash content, 10.3 per cent. Therefore it was necessary to interpolate the contractor's guarantees for these particular data. Table II indicates the general result of the test. The figures in the column, "Net Guarantee," represent the interpolations, considering the figures in the city's original specifications as par, whereas those in the column, "Actual Performance," were compiled from the records of the test.

TABLE II. COMPARISON BETWEEN ADJUSTED GUARANTEES AND ACTUAL PERFORMANCE

Item	Net Guarantee	Actual Performance	Passing Margin, in Percentage
Capacity, in wet tons per hour	1.92	2.23	16.1
Capacity, in wet tons per day	46.08	53.52	
Oil, in gallons per wet ton	9.38	7.09	24.4
Oil, in gallons per day	225	170	
Air, in cubic feet per wet ton	562	356	36.7
Air, in cubic feet per day	13,488	8,544	
Power, in kilowatt-hours per wet ton	0.88	0.28	68.2
Power, in kilowatt-hours per day	21.12	6.72	

The moisture content of the pressed screenings averaged 66.2 per cent instead of the guaranteed 70 per cent. The ash was clean and uniform and contained less than 1 per cent of organic matter. No smoke or vapor was given off from the chimney, and there was no noticeable odor at a distance of 100 ft from the plant.

On completion of the official test the trial test was at once begun. It continued until January 13, 1934. With the exception of a few minor replacements in the furnace, specifications were met and guarantees fulfilled during this test. Therefore the plant was accepted.

A factory type of building with a heavy timber frame, corrugated iron roof and sides, and large window area has been erected over the plant. Telephone and electric signal devices have been installed in order to correlate operations with the screening plant. An ejector has been added for the handling of ash. The debris from the coarse bar screens, removed by automatic rakes, is to be conveyed to the incinerator by a pneumatic pipe line, but because this material includes long rags, sticks, and coarse matter, the rolls will be by-passed and the debris discharged directly into the storage bin.

The specifications were prepared, and the installation and test operation supervised by the Division of Sewer Design, Bureau of Engineering, in the Department of Public Works of the City of Los Angeles.

The Maryland–West Virginia Boundary

A Controversy Extending Through Three Centuries and Involving the History of Three States

By LYNN PERRY
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TO trace the history of the boundary dispute between Virginia and Maryland, it is necessary to go back to the original charter for the latter state, issued by Charles I to Lord Baltimore in 1632. The first boundary difficulty was caused, not by lack of clearness in this charter, but by lack of geographical knowledge. The charter specified the "furthest fountain" of the Potomac River as the southwestern boundary point, but it was not until 1754, when Colonel Cresap made his survey, that it was definitely known which fork of the Potomac was the longer. Later, when the State of Maryland had an opportunity to assume jurisdiction over territory included in the original grant, she failed to take the necessary steps. According to Mr. Perry, indecision and delay lost her the disputed area. Many commissions were appointed and various surveys made before the matter was finally settled in 1912.

NOT only the engineer but also the historian and the antiquarian are interested in the history of the boundary dispute between Maryland and West Virginia. The facts influencing the final settlement illustrate a number of legal points with which every experienced land surveyor is familiar and recall some outstanding events in Colonial history. On the accompanying map, Fig. 1, the charter boundary of Maryland is indicated by a heavy line, the present boundary by a lighter line, and the territory awarded to her neighbors by hatching. Also, the date of each award is given.

On June 16, 1632, the original charter for Maryland was issued to Cecelius, second Baron of Baltimore, by Charles I. The territory included in this grant lay entirely within the boundaries of other charters previously granted to the Virginia Company by James I, but in 1632 Virginia was a royal colony, and from the King's decision there could be no appeal. This 1632 charter, written in Latin, bears evidence of considerable care in its preparation. The boundaries are so carefully and accurately described that it must have served as a model for some generations. According to this charter, liberally translated, the bounds begin where the fortieth parallel, north latitude, crosses the Delaware River, run west along the fortieth parallel to a point due north of the "furthest fountain," or source of the Potomac River (furthest from its mouth), thence due south to the Potomac, thence to the east following the high water mark on the south side of the Potomac River to its mouth, thence across the Chesapeake Bay to Watkins Point, thence east to the Atlantic Ocean, thence along the shore of the Atlantic Ocean and of Delaware Bay to the point of beginning.

At that time the territory west of the navigable parts of the streams flowing toward the Atlantic was entirely unexplored, and the geography of the continent was little understood. Even as late as 1670, Augustin Herman, an unusually well-informed traveler and promotor of trade routes, in his map considered the mountains in the vicinity of Cumberland to be the continental divide.

As a proper background for the controversy under consideration, a part of Virginia's Colonial history must be outlined. In 1649, Charles II, at that time a fugitive in Holland, granted to Lord Hopton, to Sir Thomas Culpeper, and to other exiled royalists a tract of land in Virginia lying between the Potomac and Rappahannock rivers and extending from Chesapeake Bay to the heads of these streams. This tract included the Colonial estates of the Washingtons, Lees, Madisons, Monroes, Masons, Marshalls, Corbins, Pendletons, and many other well-known Virginia families. During the interregnum this was merely a paper grant. After the Restoration, however, the heirs and assigns of the grantees determined to avail themselves of the rights accruing to them. The validity of the grant having been questioned, these claimants surrendered the original papers and in 1669 received in return a re-grant to the same lands, under the privy seal. This new grant, like the former one, conveyed title to the land only, gave no proprietary rights, and allowed the Virginia Colony legal jurisdiction.

In 1673 this patent was superseded by a grant to the Earl of Arlington and Lord Culpeper carrying absolute proprietary rights to the whole of Virginia and the

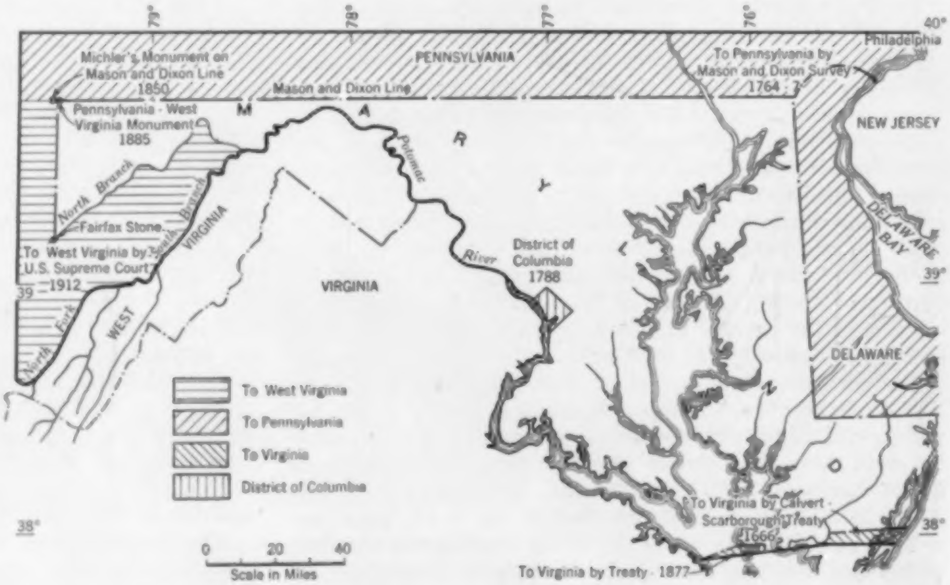

FIG. 1. MAP OF MARYLAND, SHOWING ORIGINAL AND PRESENT BOUNDARIES

Accomack Colony. The Virginians were vigorously opposed to this grant to two of the most unscrupulous courtiers of that day, and continued so. It was the most potent factor leading to Bacon's Rebellion, the outstanding feature of Virginia's Colonial history. Even after the Revolutionary War, in 1785, an attempt was made to have it canceled, but the U. S. Supreme Court ruled otherwise. Maryland, taking the instrument at its face value, was not particularly interested inasmuch as the land in question was specifically within Virginia's jurisdiction. It was not foreseen that a survey would result in nearly two hundred years of litigation. It did finally result in the extension of the boundary of Virginia and in transference to West Virginia, Virginia's successor, of a part of the original grant to Lord Baltimore.

In 1688 the title to all the Arlington–Culpeper land was held by Thomas, Lord Culpeper, and in that year James II granted him a new patent to the entire tract. At his death, it went to his only daughter, Catharine, who brought it in marriage to Thomas Fairfax, fifth Baron of Cameron, in the Scottish peerage. Lord Fairfax, determined to reap the benefit from this immense territorial possession, which had been re-granted to him by George II, in 1733 petitioned the King for commissioners to fix his boundaries. Since this territory was becoming populated by Scotch-Irish and Dutch, Virginia had made a similar petition in 1730. Three commissioners were appointed to represent the Crown and a like number to represent Virginia. Maryland was not represented on this commission because the grant called for lands in Virginia only, entirely south of the Potomac River, and Charles, fifth Baron of Baltimore, at that time Lord Proprietor of the Maryland Colony, could not see that his rights would be affected in any way. When the question arose, in 1736, which of the two branches of the Potomac River was the longer, the commissioners concluded that it was the North Branch. History does not record how this conclusion was reached, or on what information the commission based its decision. Lord Baltimore, who was the abutting freeholder and whose property was at stake, had no voice in the matter and was not even officially notified of the intention or conclusion of the commission. The report of this commission was confirmed by the King in council and by the Virginia Assembly in 1738.

THE FIRST SURVEY

Thomas, sixth Baron of Cameron, came to America in 1745, and in the following year engaged surveyors to run his property lines in accordance with the report of the commission. In complying with this duty, on October 17, 1746, the surveyors placed a stone, known as the Fairfax Stone, near the source of the North Branch of the Potomac to indicate the northwesterly corner of the Fairfax estate. In addition to limiting the Fairfax estate, this stone also fixed the southern end of the western boundary of Maryland. Shortly afterward, Lord Fairfax opened an office for the sale of land. In 1748–1749 George Washington assisted in the surveying and subdividing of the property.

Word of these proceedings reached Frederick, sixth (and last) Baron of Baltimore in 1753. In his first letter of instructions to Horatio Sharpe, the new Provincial Governor of Maryland, he protested against the conclusion of the commission, instructed the Governor to look into the matter, and to open negotiations with Lord Fairfax looking toward an adjustment of his property line and the placing of the southern end of the western boundary of Maryland at the source of the southern and longer branch of the Potomac. With these instructions, the Maryland Council called before them Col. Thomas Cresap, a well-informed settler and surveyor who lived in the extreme western part of the province. (In his diary, George Washington states that Colonel Cresap lived at the mouth of the South Branch.) Colonel Cresap informed them that, in his opinion, the South Branch was about 60 miles longer than the North Branch.

Having been apprized of these facts, Lord Fairfax probably thought that he would gain land by going farther west and remaining below the South Branch. He expressed his willingness to agree to a compromise in a letter to Governor Sharpe, but indicated that he thought the matter one which the governors of the two colonies should settle.

The following year, 1754, Colonel Cresap made a survey of both branches and sent Governor Sharpe a map. This map is remarkably accurate and complete, and shows all the information necessary for its purpose fully as well as such a map made today could do. It is evident that the Governor was prepared to take the steps necessary to secure the rights of Maryland, but the outbreak of the French and Indian War delayed the proceedings. It would have been impracticable, if not impossible, to make the necessary surveys in hostile territory during this conflict, so further action was postponed until after the Treaty of Paris in 1763.

A ROYAL PROCLAMATION

At this time, the Crown thought that a general adjustment of Colonial boundaries might be desirable or necessary. In addition it had certain obligations to the Indians created by the treaty, so a royal proclamation was issued forbidding Colonial authorities to make grants of land west of the headwaters of streams flowing into the Atlantic Ocean. At this time also, Lord Baltimore decided to reserve for himself a tract of some 10,000 acres as a baronial estate. As the site for this reservation he chose the mountains of western Maryland and gave instructions that no more grants of land should be made west of Fort Cumberland until this reservation could be laid out. Had it not been for these two occurrences, the western boundary of Maryland would probably have been settled at that time.

Robert Eden, who succeeded to the governorship of Maryland in 1769, was active in his aim to have the western boundary dispute settled. To this end he appointed, in 1771, a commission (of which Colonel Cresap was a member) to make the necessary surveys to determine the location of Maryland's western charter boundary. The commission made a very satisfactory survey and submitted a report and map showing the South Branch to be the longer and marking its source with a stone on which the letters "CLDB" were cut. In the meantime the "Western Reserve," west of Fort Cumberland, was opened to settlers against the protest of Mr. Jenifer, the agent of the Proprietor of Maryland, and in 1774 again closed on instructions from the guardians of Henry Harford, a minor, who had succeeded to the proprietorship of Maryland on the death of Frederick in 1771.

Thus the matter stood at the outbreak of the Revolutionary War. Maryland was standing on her rights to the charter boundary, which had never been revoked. Virginia was opposed to the Arlington–Culpeper grant, the only aggravating feature standing in the way of a peaceful and prompt settlement at that time.

In 1785, after the clouds of war had blown away, the Virginia Assembly took advantage of the alienage of

Denny Fairfax, devisee of Lord Fairfax, and brought suit attempting to cancel the Arlington-Culpeper grant. This effort was unsuccessful, but certain features of the original grant, such as quit-rents, ceased with the separation from England. The first constitution of Virginia (June 27, 1776) recognized Maryland's equity to her original charter boundary (Art. 21). Although the authorities of these two neighboring colonies had to arbitrate various matters from time to time, the rank and file of the people were generally friendly. Just at this time it appears that any commission could have settled the boundary question and given Maryland her full claim. But the Maryland Assembly was so anxious to maintain a friendly attitude toward its neighbor that courtesy was allowed to warp its judgment. In 1788, when Francis Deakin was employed to survey and plot lands in the "Western Reserve," and when 2,575 allotments of bounty lands for Revolutionary veterans were laid out, care was taken to keep all these allotments north of the North Branch and east of the Fairfax Stone. At that time the disputed land was in demand and was being settled. Whereas Maryland did not issue patents to this land, Virginia was not so punctilious. The consequence was that the land was taken up under Virginia patents, and new counties were organized under her jurisdiction. This was the last opportunity that Maryland ever had to recover her charter rights and assume peaceful jurisdiction of the territory between the North and South branches of the Potomac. Vacillation and delay made matters worse.

ATTEMPTS AT ARBITRATION

In 1795, Pinkney, Cooke, and Key were appointed commissioners on the part of Maryland to meet a like commission from Virginia and adjust the disputed boundary. However, Mr. Pinkney was sent on a foreign mission; Mr. Key moved out of the state; and Mr. Cooke declined to act, as did also Messrs. Carroll and Chase, who were named in the place of the first two. Again, in 1801, the Governor and Council of Maryland were authorized to appoint commissioners for the same purpose, and Messrs. Duvall, Nelson, and McDowell were appointed. The Virginia Legislature authorized commissioners also, but the commission never met and organized. The correspondence between the two governors indicates that the Virginia commissioners were authorized to arbitrate the western boundary only, whereas Governor Mercer of Maryland insisted on a survey to determine which branch of the Potomac was the longer. During these years county governments were being organized in the disputed territory under Virginia's jurisdiction, and her influence and authority there were being strengthened.

In 1818, the Maryland Assembly, realizing the circumstances, seems to have abandoned the state's claim to part of the disputed territory and authorized a commission to meet a similar commission from Virginia and run the western boundary of Maryland from the source of the North Branch to the Pennsylvania line. In 1821 the Virginia commission was instructed to run the line from the Fairfax Stone to the Pennsylvania line. However, at that time the Fairfax Stone was not known to be at the source of the North Branch; therefore the authorized point of beginning for the two commissions was considered to be different, and no action was taken when the joint commission convened in 1824. By this time some of the land lying west of the Deakin Line of 1788, and east of a line due north from the Fairfax Stone, had been taken up under Virginia letters patent; this land would have had to be re-granted to the patentees by Maryland if the commission had been able to run the western boundary north, beginning at the source of the North Branch.

In 1826, the Maryland Assembly again authorized a commission, which was not to become active until a similar commission was authorized by Virginia. It provided that in case of disagreement of the joint commission, the Governor of Delaware should be requested to name an odd member and that the decision of the commission should be final. Some of the correspondence between Governors Kent and Tyler (afterward President Tyler), of Maryland and Virginia, respectively, has been preserved, but the Virginia Assembly did not authorize the commission.

In McMahon's *History of Maryland*, published in 1831, there is considerable reference to this controversy. This book was widely circulated and inspired the next General Assembly (1832) of Maryland to take up the

Courtesy U. S. Coast and Geodetic Survey

GRANITE MONUMENT ERECTED IN 1885 BY THE STATES OF PENNSYLVANIA AND WEST VIRGINIA

Placed to Mark the Corner Common to These States and to Maryland, Which State However Had No Part in Locating It. It Is near the True Corner, According to the 1912 Decree of the U. S. Supreme Court

matter once more. The act of 1832 authorized a commission similar to that of 1826. The Virginia Assembly, however, maintained the same position it had taken in 1821; that is, the Virginia commission was instructed to begin its survey at the Fairfax Stone and run north to the Pennsylvania line. It was also instructed to organize and run the line with or without the cooperation of the Maryland commission. These instructions in the Virginia act apparently astonished the citizens of Maryland, who seem to have considered them not only unfriendly but quite discourteous. In the light of a better prospective, this should not have been so. The leaders of the Maryland Assembly certainly were familiar with the conditions on the ground, with the U. S. Supreme Court decision in relation to the Arlington–Culpeper grant, and with the position Virginia had taken in 1821. But they did not appear to realize that Virginia could not recede from that position in the light of all of the facts surrounding the case. Nevertheless, the Maryland Assembly, in 1834, authorized the Attorney-General of the state to start proceedings in the Supreme Court of the United States and secure final settlement of the boundary. No doubt that official was perfectly familiar with the U. S. Supreme Court decision of 1785 confirming the rights of the Fairfax heirs: no doubt he had little hope of securing a decree in any way contrary to that decision, but the demand of the multitude had to be met.

At the next session of the Virginia Assembly, Governor Tazewell, in his message to that body, referred to these proceedings as an unfriendly and menacing attitude on the part of a neighboring commonwealth. When his message was brought to the attention of the Maryland authorities, through the press, a committee of the Maryland Assembly recommended that, since the court proceedings were standing in the way of an amicable settlement, they should be withdrawn. This was done and the matter was allowed to slumber for another twenty years.

Maryland had refrained from issuing grants for land west of the block laid out by Deakin in 1788 for Revolutionary War veterans. The western boundary of these grants was irregular but everywhere well within a line extending due north from the Fairfax Stone. The boundary of these grants had therefore done service as the western boundary of Maryland for over half a century. During this time, land had been granted by letters patent under Virginia laws all the way up to these grants. Neighbors were continually having disputes about citizenship and tax levies. In 1852, the Maryland Assembly passed an act reciting these facts and requesting the Governor to take the matter up with the Virginia executive. He was authorized to appoint a commissioner to run and mark the state boundary north from the Fairfax Stone if Virginia should do likewise. In 1853 Virginia responded by appointing a commissioner with similar powers, and it again looked as though the matter would be settled. A subsequent act on the part of the Virginia Legislature provided for a joint commission to trace not only the boundary from the Fairfax Stone north but also the line from the mouth of the Potomac (Smith's Point) to the Atlantic Ocean.

Lieutenant Michler's Survey

The commissioners appointed were T. J. Lee for Maryland and Angus W. McDonald for Virginia. Inasmuch as Mr. Lee was not authorized to run the line across the Chesapeake Bay and the Eastern Shore Peninsula, the commissioners decided, pending further instructions, to run the line from the Fairfax Stone north to the Pennsylvania line. To this end, they secured the services of Lieut. N. Michler, of the U. S. Corps of Topographical Engineers, an experienced and capable officer, and organized a surveying party, which ran the line in 1859. Lieutenant Michler's field notes were complete and carefully kept. He had no difficulty in finding and identifying the point of beginning, the Fairfax Stone, placed in 1746.

This stone is described in the field book of Thomas Lewis, one of the surveyors who made the original Fairfax survey, as follows:

> October 23, 1746. Returned to the spring where we made the following marks:—on another Beach WB WR 1746 Y3—a stone by the corner pine marked FX, on a Beach marked AC.

In 1859, Michler described the stone thus:

> The initial point of the work, the Fairfax Stone, stands on the spot encircled by several small streams flowing from springs about

Courtesy U. S. Coast and Geodetic Survey

The Michler Stone, Erected in 1860 by Lieutenant Michler at North End of Michler Line

This Stone Is About a Mile West of the Stone Set in 1885 by Pennsylvania and West Virginia on the Mason and Dixon Line

it. It consists of a rough piece of sandstone, indifferent and friable, planted to a depth of a few feet in the ground and rising a foot or more above the surface; shapeless in form, it would scarcely attract the attention of the passerby. The finding of it was without difficulty, and its recognition and identification by the inscription FX, now almost obliterated by the corroding action of water and air. In order not to disturb the stone, the first observatory was built immediately in the rear (south) of it.

Michler's boundary marker was about 4 ft high, of hewn stones, the top ones conical. The original Fairfax Stone was destroyed by vandals and carried away about 1883, leaving the Michler monument the only boundary marker for some years. It was on the Appalachian divide about a quarter of a mile north of the Western Maryland Railroad and about half a mile northwest of Fairfax Station. Most of the land, once densely wooded, is now covered with a thick undergrowth of brush and briars and belongs to the Davis Coal and Coke Company. The northern end of the "Michler Line" was marked by a granite monument (shown in a photograph) called the "Michler Stone." This is on the Mason and Dixon Line.

On March 6, 1860, the Michler Line was ratified by the Maryland Assembly. The Virginia Assembly of 1859–1860 passed two acts relating to the subject, neither of which ratified the line. One provided for suitable monuments to mark the line, and the other authorized further research on the subject of Virginia's boundaries with North Carolina, Tennessee, and Maryland. The war between the states halted the proceedings.

WEST VIRGINIA INHERITS THE CONTROVERSY

West Virginia was authorized by act of Congress of December 31, 1862 (the original name proposed for this state was Kanawha), and its boundaries were designated by county lines. Later, in 1866, it was enlarged by the addition of other Virginia counties. As West Virginia then embraced all the territory concerned in the boundary controversy, that state inherited the controversy also.

In 1868 the West Virginia Legislature authorized the Governor to communicate with the Maryland authorities with a view to re-running the boundary, taking the stand that as Virginia had not ratified the Michler Line, West Virginia was not bound by it. In 1886, the Maryland Assembly instructed the Governor of that state to take the matter up with the Governor of West Virginia, to call his attention to the state of the controversy, and to request him to lay the matter before the West Virginia Legislature. This was done, and the following year (1887) this legislature ratified the Michler Line on condition that the Maryland Assembly pass an act confirming all Virginia patents in the strip of land lying between the Michler Line and Deakin's Line. As some of this land had been taken up under Virginia patents and some under Maryland patents, such an act would place the Maryland Assembly in the position of abrogating some of its own grants in order to gain possession of a strip of land three-quarters of a mile wide and 37 miles long. Such a resolution was never passed, and the matter slumbered for another decade.

THE LINE FINALLY ESTABLISHED AND MARKED BY CONCRETE MONUMENTS

A resurvey made in 1897 and court proceedings instituted by Maryland once more brought the issue to a head. The U. S. Supreme Court, in a decree entered May 31, 1910, appointed Julius K. Monroe, Samuel S. Gannett, and W. McCulloh Brown as commissioners to re-run the old Deakin Line as the true, proper, and final boundary between Maryland and West Virginia. Field work was begun on July 12 and finished in October of the same year.

This commission replaced Michler's monument at the site of the Fairfax Stone with another, more substantial and more permanent in its construction. It began the boundary 3,989.13 ft N. 0°56′00″ E. of this place, at a point on the low-water mark on the south side of the North Branch of the Potomac and proceeded north between the lands granted under Maryland patents on the east and those under Virginia patents on the west, 19 courses, 189,798.97 ft (35.95 miles), to the Pennsylvania line. Some trees called for by Michler (1859) and Deakin (1788) were found and identified by the proper number of annual rings of growth over the old blaze scar. They were replaced by more permanent monuments. Some of these courses ran nearly due east and west for a distance as much as 971 ft, but the average bearing was slightly east of north. The latitude and longitude of the point of beginning are 39°12′21.34″ N., 79°29′14.67″ W., and those of the northern end, 39°43′ 15.88″ N., 79°28′37.72″ W. On small maps this line appears to follow the 79°30′ meridian.

At each angle and at other important points, 34 in all, the line was monumented. The principal monuments are of 1:2½ cement-sand mortar, 22 in. square at the base, tapering to 10 in. square at the top. They are 4 ft in height, with corners beveled 1½ in., and have a pyramidal top about 4 in. high above the square section. They are set on concrete foundations 3½ ft square by 2½ ft deep, or deeper, depending on the conditions necessary to ensure stability. The monuments are numbered from 1 to 34 consecutively northward. On the south side they bear the number, 2¾ in. high, and the names of the commissioners in 1-in. letters; on the east and west sides, respectively, the letters "Md." and "W.Va." in 3½-in. letters; and on the north side, the date, 1910. These inscriptions were molded in the monuments by attaching reversed, bevel-faced characters to the inside of the forms.

SUPREME COURT ISSUES FINAL DECREE

Thus has the boundary been fixed and monumented as well and as permanently as is practicable. The final words of the story were written on May 27, 1912, when the U. S. Supreme Court decreed this line, ordered by it and executed by its commissioners, to be the true boundary line between the said states. Maryland retains all the territory over which she ever exerted jurisdiction. West Virginia was awarded territory within the charter boundary of Maryland because her predecessor had assumed and maintained jurisdiction.

Much of the material for this article has been compiled from the decree of the U. S. Supreme Court, October term, 1911 (225 U. S.), and to the brief and the addenda to the brief, on the part of Maryland, October term, 1909. Valuable information has also been obtained from Wise's *Early History of the Eastern Shore of Virginia;* McMahan's *History of Maryland;* Whealton's *Maryland and Virginia Boundary Controversy;* the *Report of the Committee on the Western Boundary of Maryland* and other papers of the Maryland Historical Society; *History of Virginia*, by Campbell; *History of Virginia*, by Cooke; and *History of Western Maryland*, by Scharf. Reference has been made also to the *Proceedings of the Council of Maryland* and to the U. S. Geological Survey *Bulletin No. 817*, as well as to other publications of the Survey.

Gravity Bulkheads and Cellular Cofferdams

Essential Theory of Design When Steel Sheet Piling Is Used

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Steel sheet piling has proved a useful material for the construction of gravity-type bulkheads and cofferdams. The width between the parallel walls of the bulkhead must be such that the structure will not overturn; the weight of the whole, or the penetration of the piles, must be sufficient to prevent sliding; and the walls themselves must be so tied together as to keep the piles aligned. In the circular-celled cofferdam, the diameter of the cells is a function of the depth of the water. In this article Mr. Pennoyer explains the design features of three common types of construction, in which the boxes or cells are filled with suitable sand or stone to form a continuous self-supporting wall. He takes the design of such structures out of the rule-of-thumb class and bases it on rational analysis.

A steel sheet-piling bulkhead of the gravity type consists of two parallel walls of piling, suitably tied to each other, and of soil or broken stone confined between these walls. The tendency of the structure to overturn (or to rotate), to become distorted, and to slide in resisting the pressure of external earth or water or other forces, is prevented by the weight of the confined material and its frictional value.

Such a bulkhead is properly designed in five steps, by determining: (1) the lateral loads due to the external earth or water, with the resultant load and its location; (2) the proper width of the structure to resist overturning or rotation; (3) the factor of safety against sliding; (4) the internal shear in the confined material; and (5) the bending moment due to the internal lateral loads caused by the confined material, to be resisted by wales and tie-rods if the external walls are in a straight line, or by the tension strength of the interlocks if the walls are arched.

In a previous article, "Design of Steel Sheet-Piling Bulkheads," in the November 1933 issue of Civil Engineering, I described the development of the external lateral earth loads and their distribution. The following symbols there used will be repeated for convenience:

p_e = increment of horizontal liquid pressure of dry or moist earth, in pounds per square foot

$p_{e\ in\ w}$ = increment of horizontal liquid pressure of submerged earth, in pounds per square foot

p_w = the increment of the hydrostatic pressure, in pounds per square foot

= w_w, weight of water, in pounds per cubic foot

p_{comb} = the increment of the lateral liquid pressure of combined submerged earth and water, in pounds per square foot

= $p_{e\ in\ w} + p_w$

w_e = weight of dry or moist earth, in pounds per cubic foot

$w_{e\ in\ w}$ = weight of submerged earth, in pounds per cubic foot

ϕ = angle of repose or internal friction of dry or moist earth, in degrees

ϕ' = angle of repose of submerged earth, in degrees

The Coulomb formula only will be used, in which the weight of the earth is resolved into an equivalent horizontal lateral pressure, as follows:

$$p_e = w_e \tan^2 (45° - {}^1/_2\, \phi) \quad [1]$$

$$p_{e\ in\ w} = w_{e\ in\ w} \tan^2 (45° - {}^1/_2\, \phi') \quad [2]$$

A typical problem is illustrated in Fig. 1, in which all heights are in feet. The distribution of the external lateral loads is illustrated by the triangles and the rectangle; and the areas representing the individual lateral loads, P_1, P_2, and P_3, are in pounds per foot of width. Then

$$P_1 = \frac{p_e (h')^2}{2} \quad [3]$$

$$P_2 = p_e h' h'' \quad [4]$$

$$P_3 = \frac{p_{e\ in\ w} (h'')^2}{2} \quad [5]$$

and the total resultant lateral load, P, is

$$P = P_1 + P_2 + P_3 \quad [6]$$

Individual loads act through the respective centers of gravity of the individual areas, which in the case of the

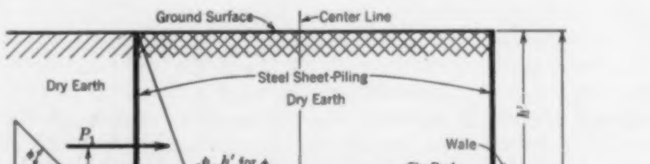

Fig. 1. Loads and Forces Acting on a Gravity-Type Bulkhead

triangles are assumed to be above the base a distance of one-third the altitude, and in the case of the rectangle, a distance of one-half the altitude. The location of the point of application, y (in feet), above the bottom is found by equating moments about the bottom. Then

$$y = \frac{P_1\left(h'' + \frac{h'}{3}\right) + P_2\left(\frac{h''}{2}\right) + P_3\left(\frac{h''}{3}\right)}{P} \qquad [7]$$

The tendency to overturn or rotate due to the external lateral load, P, is resisted entirely by the weight of the confined soil or broken stone retained between the parallel walls of the sheet piling, acting vertically downwards through the center of gravity of the bulkhead. If x is the unknown width of the bulkhead in feet, then the center of gravity lies on a vertical line located at $\frac{x}{2}$.

Tension at the heel of the bulkhead is zero if the resultant, R, of the external lateral load, P, tending to cause overturning, and the weight, W, of the confined material resisting rotation intersects the base line, ab (Fig. 1), within its middle third. The bulkhead is then also safe against overturning or rotation.

The weight, W (in pounds) of a section of the bulkhead of width x (in feet) and thickness of 1 ft is:

$$W = x(w_e\, h' + w_{e\ in\ w}\, h'') \qquad [8]$$

$$\text{Then } \tan\theta = \frac{P}{W} = \frac{x}{6} \div y \text{ and } x = \frac{6\,Py}{W}$$

or

$$x = \sqrt{\frac{6\,Py}{w_e h' + w_{e\ in\ w} h''}} \qquad [9]$$

In this equation, the denominator is the weight of a column of earth or rock 1 ft square and of height h. If x, in feet, is at least equal to this ratio, then the bulkhead is safe against overturning.

The dimension y is the distance from the point of application of the resultant of the external loads to the point of rotation of the bulkhead. This point of rotation is at the elevation of the bottom only when it is rock. If the bottom is of soil, the point of rotation is at a distance below the bottom depending on the stability and passive resistance value of the soil.

If the surface below ab in Fig. 1 is bedrock, into which the sheet piling cannot be driven, the friction of the soil on this rock is the only resistance to sliding. Safety against sliding is assured as long as the tangent of the angle between the resultant, R, and the vertical, θ (Fig. 1), is less than f, the coefficient of friction of the soil on rock. Obviously, the coefficient of friction between the rock and the soil must be known, but information on it is rather fragmentary. Indebtedness is acknowledged to Brennecks-Lohmeyer for Table I, giving the coefficients of friction for various types of soil on rock.

If the bulkhead is built on soil, and if for construction reasons it is not necessary to drive the sheet piling below the surface ab, the coefficient of internal friction of the soil must exceed θ, provided the soil above and below the plane ab is of like composition and density. However, if the soil above ab is loose fill or of different composition from the soil below, no known data are available.

Usually, when gravity bulkheads are built on soil, the sheet piling is driven to any necessary depth below the bottom, as illustrated by the dotted line below point O in Fig. 1, thus preventing sliding by the passive resistance of the soil. Internal friction of the soil at ab can then be ignored as far as resistance to sliding is concerned.

TABLE I. COEFFICIENTS OF FRICTION OF SOILS ON VARIOUS TYPES OF ROCK

DESCRIPTION OF SOIL	CLASSES OF ROCK				
	Rough Granite	Smooth Sand-stone	Rough Sand-stone	Smooth Masonry	Rough Masonry
Gravel and sand:					
Dry	0.54	....	....	....	....
Wet	0.48	....	....	....	....
Fine sand:					
Dry	0.70	....	....	....	....
Wet	0.53	....	....	....	....
Fluid slime	....	....	....	0.05	0.10
Firmer slime	....	....	....	0.10	0.20
Wet clay and loam	....	....	....	0.20	0.30
Dry sand	....	....	....	0.60	0.70
Wet sand	....	....	....	0.30	....
Dry gravel	....	0.57	0.61	0.40	0.50
Wet gravel	....	0.60	0.62	0.40	0.50

Next determine the depth, z (in feet), to which the sheet piling should be driven to prevent sliding due to the external load, P. This sliding is resisted by the passive pressure of the soil into which the piling is driven. According to Coulomb's law, this passive pressure is:

$$p_p = w_e \tan^2 (45° + {}^1/_2\,\phi) \qquad [10]$$

in which p_p is the increment of equivalent liquid passive resistance of the earth, in pounds per square foot. If

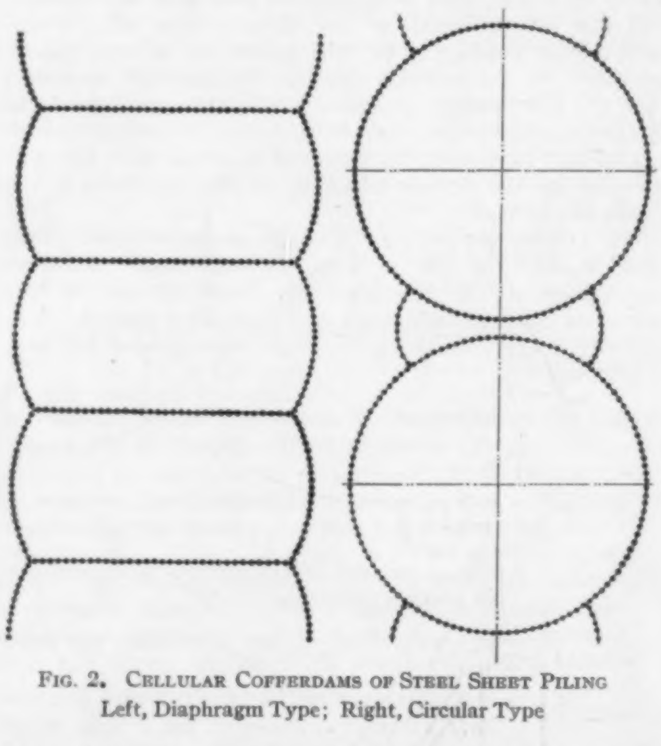

FIG. 2. CELLULAR COFFERDAMS OF STEEL SHEET PILING
Left, Diaphragm Type; Right, Circular Type

the earth is submerged, use $w_{e\ in\ w}$ instead of w_e. Then $p_p \frac{z^2}{2}$ must be equal to or greater than P, and

$$z \gtrless \sqrt{\frac{2P}{p_p}} \qquad [11]$$

If z equals or exceeds this ratio, the bulkhead is safe against sliding.

In cases where the plane ab is a rock surface, the resistance of the toe of the steel sheet piling against sliding along this surface is an unknown factor of safety. That

this resistance is considerable is well known, because it is rarely that even bedrock does not have a few inches of rotted rock at its surface into which the steel can penetrate. Also, if the soil in contact with the rock at the plane *ab* is stable sand, gravel, clay, or broken stone, except in extraordinary cases of very heavy external loads, the friction is entirely sufficient to resist sliding with an ample factor of safety.

Internal shear in the material confined between the two walls of the sheet piling is especially important if this material is unstable fill. According to Hooke's law, in an elastic solid the horizontal and vertical shear at any point are equal. The external pressure is assumed as uniform along the bulkhead. The maximum internal shear occurs along the vertical center line and is therefore maximum at point c, the intersection of the center line with plane *ab* (Fig. 1). Also, along any horizontal plane through the bulkhead, the total horizontal internal shear is equal to the total lateral load above this plane and varies from maximum at the center line of the plane to zero at the piling.

Applying the law of mechanics governing shear, the vertical and horizontal shear, s, in pounds per square foot, at point c is:

$$s = \frac{P\left(\frac{x}{4}\right)\left(\frac{x}{2}\right)}{\frac{x^3}{12}} = \frac{3}{2}\,\frac{P}{x} \qquad [12]$$

In which P is the total lateral external load above point c, in pounds per foot of length, and x is the width of the bulkhead, in feet. Also, the internal shear at the intersection of any horizontal plane with the center line is equal to this ratio (Equation 12) when P is the total lateral load above the plane.

If the internal shear were plotted along the center line as an axis, and if the increment of the lateral force P were constant, the curve would be parabolic in form, the value being maximum at the plane *ab* and zero at the top. In most bulkheads this increment will vary, but it will maintain an approximately parabolic form. Using ϕ as the angle of internal resistance of the soil, the actual shear resistance, s_e, at plane *ab* (Fig. 1), in pounds per square foot is therefore:

$$s_e = (w_e h' + w_{e\ in\ w} h'') \tan \phi' \qquad [13]$$

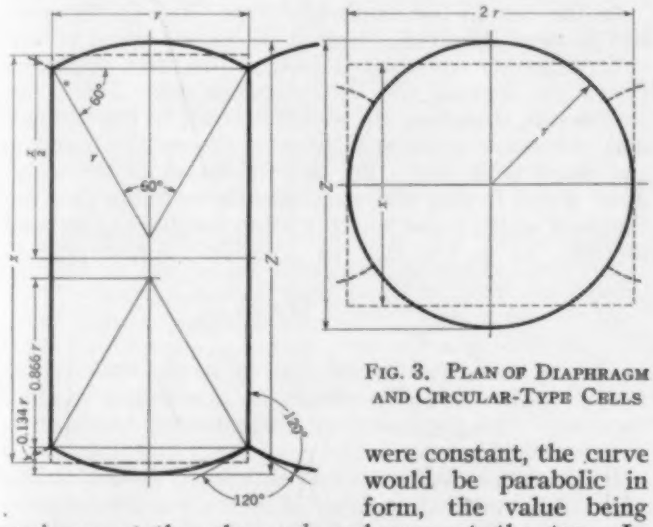

FIG. 3. PLAN OF DIAPHRAGM AND CIRCULAR-TYPE CELLS

If the value of this shear resistance is plotted on the same vertical axis as the actual internal shear, and if the shear resistance of the confined material is uniform from top to bottom, the resulting graph will be a triangle. Since most bulkheads will contain strata of different shear resistance values, the line representing shear resistance plotted along the center line will be broken at the elevations of these strata, but will be approximately triangular in form. The shear resistance should of course exceed the internal shear caused by the external load at point c by as large a factor of safety as possible. If the bulkhead is to contain material with insufficient shear resistance, the bulkhead must be wider than that necessary for stability against overturning. If the bulkhead should fail in shear, the two walls would rotate outward in the form of a parallelogram.

STEEL SHEET-PILING COFFERDAM OF CIRCULAR TYPE IN A RIVER
For the Construction of a Dam

BENDING MOMENT AND LOADS ON TIE-RODS AND WALES

The bending moment in the outside wall due to the lateral load of the confined earth and the loads on the wales and tie-rods are calculated exactly as outlined in my previous article in the November 1933 issue, "Details of Steel Sheet-Piling Bulkheads." The internal lateral load of the confined material on the outside wall has no relation to the external load tending to cause sliding or overturning and is a separate study. In the special case, Fig. 1, the external and internal loads happen to be the same.

When the plane *ab* is bedrock, provision must be made to prevent the toe of the sheet piling on the outside wall from moving outward as a result of internal pressure. This is done by either cutting a trench in the rock, which is expensive, or blasting holes at set intervals for the insertion of master piles or dowels.

Another successful method of holding the toe of the piling is to fasten every alternate sheet pile to the one directly across from it in the opposite wall by means of a tie-rod. These rods are inserted in holes in the piles before they are driven, and each two piles thus connected are driven to grade simultaneously.

When the plane *ab* is soil, the sheet piles are driven to sufficient depth in the outside wall to prevent lateral movement (the determination of this depth was outlined in my previous article in the November 1933 number). All the preceding remarks of course apply only to bulkheads so constructed that the internal bursting loads are resisted by the beam strength of the external walls and not to cellular bulkheads with arched external walls and internal sheet-piling walls extending from top to bottom, where the sheet piling is entirely in tension.

A typical gravity-type bulkhead is the cellular cofferdam. Such a design is necessary when foundations or

other construction are in very deep water, are carried to great depths, or are so huge that the single-wall cofferdam of steel sheet piling, internally braced, is not practical. For a cellular cofferdam the sheet piling is driven in the form of a continuous series of connected cells, filled with

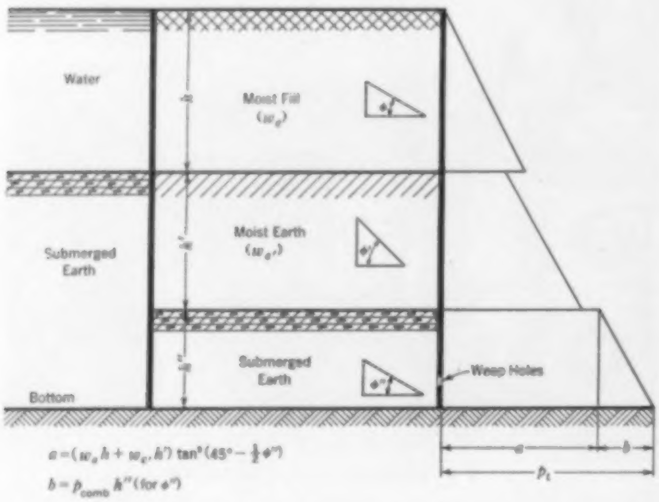

FIG. 4. DIAGRAM FOR DETERMINING THE BURSTING PRESSURE IN A CELLULAR COFFERDAM

broken rock or soil, so that the weight of the contents affords stability against external forces. In the "diaphragm type" of cellular bulkhead, illustrated in Fig. 2, the outside walls are driven in a series of arcs connected by fabricated Y-pieces. The stems of these Y-pieces are linked across the bulkhead by continuous diaphragm walls of sheet piling. In the same figure there is a sketch of the circular type of cellular cofferdam, consisting of a series of complete circles connected by short arcs through fabricated T-pieces. The diaphragm type requires less sheet piling, but the circular type is advantageous in that each circle can be completely filled, and therefore made stable, immediately after the piles are driven. The diaphragm type must be filled in stages, keeping the height of the fill in adjoining cells as nearly as possible at the same height to avoid distortion of the diaphragm walls. However, these diaphragms can be slightly arched to overcome this difficulty. A disadvantage of this expedient, however, is the doubtful stresses created at the Y-connections, which then connect three arcs.

Although the usual use of the cellular bulkhead has been for cofferdams, which are temporary structures, the principle is now being applied to permanent construction such as wharves, retaining walls, and breakwaters. The U. S. Navy has built a wharf at Coco Solo, C.Z., of the diaphragm type illustrated in Fig. 2. The U. S. Engineers commenced the construction of huge cellular breakwaters in the spring of 1934, located at Calumet Harbor, South Chicago, and Port Washington, near Milwaukee, Wis. This type is also being carefully studied for all other breakwaters contemplated on the Lakes. Such construction is economical because the sheet piling can be light in weight, strength in tension, and not section modulus, being the only consideration.

The difference between the cellular bulkhead and the straight-wall gravity type is that, in the former, the outside walls are driven in arcs or circles, and the wales and tie-rods are replaced by continuous connecting walls of sheet piling. This places the sheet piling entirely in tension, and any lateral movement of the external walls is resisted by the integral internal walls. In the design of the cellular type it is important to understand in what ways it is similar to the gravity type and in what ways it is different.

First, the external forces, their resultant, and their point of application are developed as previously outlined. The proper width for stability against overturning or rotation is first calculated as though the cofferdam were a rectangular box, the same as for the straight-wall gravity type.

Since the external walls of the cellular type are curved, the width must be increased in order that there may be a sufficient quantity of fill to make it as effective against overturning as the rectangular type. If a horizontal section through the cellular bulkhead has the same section modulus as the rectangle, then both are equally resistant to overturning under the same external force. The proper width for the rectangular wall having been determined, the problem is to find the diameter, or width (Z), for the cellular type that will give it the same section modulus.

In Fig. 3, x represents the width, in feet, of a rectangular bulkhead necessary for its stability, as determined by Equation 9; and r represents the unknown radius, in feet, of a circle having the same section modulus as the rectangle. The section modulus of the rectangle is $\frac{2rx^2}{6}$, or $\frac{rx^2}{3}$, and that of the circle is $0.7854\, r^3$.

Then $0.7854\, r^2 = \frac{x^2}{3}$ and

$$r = \sqrt{\frac{x^2}{2.3562}} \quad \ldots\ldots\ldots\ldots\ldots [14]$$

If the circles in the cells are given this radius, the supporting value of the volume contained between the short arcs connecting the circles is ignored, since it is an error on the safe side. In general, if the diameter, Z, is taken as the width, x, of the rectangle divided by 0.80, the bulkhead will have a section modulus about equal to that of the rectangle of width x and length $2r$ in Fig. 3.

In the case of the diaphragm type (Fig. 3), the problem is more involved. Except in special cases, cellular cofferdams are constructed so that the three angles between the legs of the Y-connections are all 120 deg. In the cell, therefore, the chord is equal to the radius r, and the angle contained between the radii connecting the chord is 60 deg. The determination of the width, Z, of a cell having the same section modulus as a rectangle of width x and length r is too involved to be made directly, as in the case of a circle. Approximately,

$$Z = x + 0.16\, r, \text{ or } \frac{x}{0.9}$$

The section modulus of the rectangle enclosed between the two opposite chords is then determined and combined with the moduli of the segments between the chords and the arcs.

In this connection, it is useful to know that a circular segment subtending an angle of 60 deg has the following properties: its area is $0.906\, r^2$; the distance from the chord to its center of gravity is $0.0537 r$; its moment of inertia about an axis parallel to the chord though the center of gravity equals $0.00013\, r^4$; and its moment of inertia about the chord is $0.00039\, r^4$.

Since the external walls of a cellular bulkhead are driven in either arcs or circles, the retained material exerts a bursting pressure which is resisted by the strength in tension of the sheet piling and the interlocks. No practical interlock has yet been designed to develop the full strength of a $^3/_8$-in. thickness of steel in the

sheet-piling web; therefore the discussion that follows will deal only with the interlocks, the part of the piling having the least strength.

A case in which the external and internal earth pressures differ and must be studied separately is illustrated in Fig. 4. This example is particularly applicable to cellular cofferdams constructed to unwater an area or to protect deep excavation. These generally contain a fill having a different density from the water or earth outside.

Manufacturers of steel sheet piling specify the minimum ultimate tension strength which they are willing to guarantee for the various sections they roll. After the proper factor of safety is decided upon, the guaranteed minimum ultimate strength in tension is divided by it, the result being the maximum working stress to be used, designated hereafter as f, in pounds per linear inch.

Whether the cellular bulkhead is built on rock or soil, the maximum bursting pressure within it occurs at the bottom and is derived as illustrated in Fig. 4. Let p_i be this lateral bursting pressure, in pounds per square foot.

The unit bursting pressure, p_i, is the same for a complete circle as for any part of it. By reference to Fig. 5, the tension in the interlocks, t, in pounds per linear inch, may be expressed as follows:

$$t = \frac{p_i r}{12} \qquad [15]$$

Substituting f, the maximum safe working tension in the interlocks, for t, the maximum allowable radius of the arc, in feet, is found to be:

$$r_{max} = \frac{12 f}{p_i} \qquad [16]$$

Again from Fig. 5,

$$\cos \alpha = 1 - \frac{d}{r_{max}} \qquad [17]$$

and

$$c = 2 r_{max} \sin \alpha \qquad [18]$$

in which

d = rise of the arc, in feet

c = length of the chord, in feet

α = chord-arc angle, in degrees

If the bulkhead or cofferdam is of the circular type, illustrated in Fig. 3, the maximum radius of the circle, in feet, without exceeding the safe stress (f) of the interlocks in tension is calculated from Equation 16. If the bulkhead is of the diaphragm type, the maximum radius of the arc is likewise obtained from Equation 16. For bulkheads other than cofferdams, such as retaining walls, breakwaters, and wharves, and sometimes for very shallow cofferdams, it is generally advantageous to stress the interlocks in the arcs to their safe limit, f, in working stress, or in other words to use r_{max} for the radius of the arc. Further, for various reasons the maximum desirable rise, d, in the arcs of such bulkheads is limited. This limit, in turn, determines the maximum chord of the arc, c, in order not to exceed the maximum allowable radius, r_{max}.

If the maximum value for d is selected and substituted in Equation 17, the chord-arc angle can be obtained. Then, by solving Equation 18, the maximum chord, c, is determined. The actual dimensions of the arc will have to be calculated from the widths of the sheet piling in the arcs, taking into account the leg dimensions of the connecting Y-pieces. If a maximum chord is first selected, α is determined from Equation 18 and, in turn, d from Equation 17.

The maximum practical value for α is 30 deg, making each angle contained between the legs of the Y-pieces 120 deg. The chord c is then equal to r, and d equals 0.13 r. With rare exceptions, cellular cofferdams of the diaphragm type are always constructed with Y-pieces having these angles. However, for construction reasons, the maximum safe chord length is not always used, in which case lower tension stresses in the interlocks result.

In the leg of the Y-connection, the tension, t_d, is the resultant of the tensions in the two connecting arc walls. When α equals 30 deg, the tension in each leg of a Y-piece is equal to

$$t_d = t = \frac{p_i r}{12} \qquad [19]$$

As long as α is less than 30 deg, the tension in the diaphragm interlock, t_d, is always less than t, the tension in the arc-wall interlock of the Y-connection.

In the circular type of cofferdam, the arcs connecting the circles should be kept as short as possible (generally from 5 to 7 sheet piles), because the tension is transmitted into the circles through the T-pieces in such a manner that the actual stresses in the interlocks are difficult to determine.

SLIDING AND INTERNAL SHEAR IN CELLULAR BULKHEADS

The safety of a cellular bulkhead against sliding is determined in the same way as that of a box-type bulkhead. It is safe to divide the total external lateral force per unit length by the width, x, of a rectangular bulkhead having the same resistance to overturning in order to obtain the average shear along the plane ab (Fig. 1).

In the case of a cellular cofferdam built on soil, to be unwatered on the inside, which retains water on the outside, the sheet piling in the outside wall should be driven deep enough to cut off any serious flow of water under the cofferdam and to prevent the accumulation of hydrostatic pressure underneath the cofferdam tending to overturn it. This deeper wall of sheet piling should also have sufficient penetration to prevent sliding if the cofferdam is vulnerable in this respect. The sheet piling in the inside wall, that is, on the unwatered side, need only penetrate the soil to a sufficient depth to retain the fill inside the cells when the excavation has reached the lowest level. Weep holes should be placed in the sheet piles in the inside walls to drain off any water that may accumulate inside the cells. This is an insurance that the fill will not be lightened by the buoyant effect of the water, and also that the safe strength of the interlocks in tension will not be exceeded.

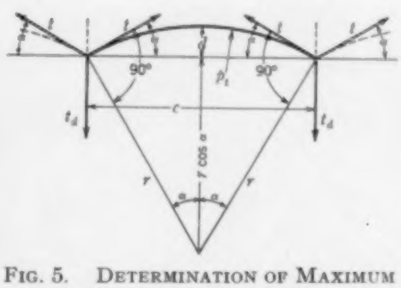

FIG. 5. DETERMINATION OF MAXIMUM TENSION IN INTERLOCKS

Internal shear is investigated according to the method outlined in the development of Equation 13. The steel diaphragm walls also resist lateral deformation of the bulkhead through friction in the interlocks, which introduces an indeterminate factor of safety in the calculation of internal shear.

Constructing the Bouquet Canyon Pipe Line

Large All-Welded Conduit in Rough Country Serves Dual Purpose

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An unusual feature of the Bouquet Canyon Reservoir, recently completed, is that 33,000 acre-ft of water from the Owens River Aqueduct can be conducted to it for storage and withdrawn from it as needed through the same pipe. Located in unbelievably precipitous country, this 3½-mile pipe line includes one inverted siphon with a static head of 870 ft. Here the pipe was laid on slopes of well over 100 per cent. The pipe is from 7 to 8 ft in diameter, is arc-welded throughout, is supported on concrete and structural steel piers, and is carefully coated inside and out with protective paint selected after many comparative tests. The outside priming coat of synthetic red paint was found to reduce the maximum temperature inside the pipe during construction about 30 deg—enough to make conditions bearable for the field welders in the summer. Special steel carriages straddling the pipe line were developed by the city's engineers and greatly facilitated field erection. Water was turned into the conduit on March 28, 1934.

A GROWING demand for water by the City of Los Angeles has required the construction of an additional storage reservoir in Bouquet Canyon, which is about four miles east of the Los Angeles–Owens River Aqueduct and about 50 miles north of the city. The newly constructed reservoir is connected to the aqueduct by a steel pressure pipe 3½ miles long, which traverses an exceedingly rough terrain. By reference to Fig. 1 it will be noted that the pipe line has the dual function of conducting water to storage in the Bouquet Canyon Reservoir when the entire flow from the Fairmont Reservoir is not needed and of returning it through the same pipe line as demand requires. Its connection to two of the penstocks of the San Francisquito Power Plant No. 1 also puts water into storage during off-peak demand, for the later generation of power. The series of pressure tunnels from Fairmont Reservoir has a maximum capacity of 1,000 cu ft per sec. The hydraulic gradient from the Fairmont Reservoir to the Bouquet Canyon Reservoir is such that the pipe line will carry a minimum of 300 cu ft per sec to storage in the latter. From storage in the Bouquet Canyon Reservoir at least 450 cu ft per sec may be withdrawn to the power plant and the city.

Of the 3½ miles of conduit, 18,000 ft consists of arc-welded steel pipe, and the remaining 1,000 ft is concrete-lined pressure tunnel. The tunnel, which connects with the reservoir gate tower through a ridge, has a finished diameter of 96 in. and a reinforced concrete lining 12 in. thick. The reinforcing steel in the tunnel lining was connected to that in the reservoir gate tower, and the concrete was poured so as to form a monolithic connection between tower and tunnel.

An idea of the roughness of the country traversed by the pipe line may be gained by a consideration of some of the problems involved. In making the pipe tangents as long as possible, and thus reducing the number of bends in the line, cuts to a maximum depth of about 20 ft were made through the ridges, and the pipe was supported on steel bents to a maximum height of a little

Bouquet Canyon Pipe Line Traverses Difficult Terrain

Left, Steel Carriage Aids in Erecting Pipe Across San Francisquito Canyon; Right, Looking West from the Tunnel Outlet

over 20 ft across some of the canyons. Nevertheless, there are 84 bends in the line. The average length of the pipe tangents is only 213 ft, and the longest is 700 ft.

The pipe varies in size from 80 in. in outside diameter and $1^1/_{16}$ in. in shell thickness at the bottom of the San Francisquito Canyon, where the hydrostatic head is 870 ft, to 94 in. in outside diameter and $^3/_8$ in. in thickness at higher elevations. The pipe was manufactured in Los Angeles and was sand-blasted, painted, placed, and welded in the field by the forces of the Bureau of Water Works and Supply, City of Los Angeles.

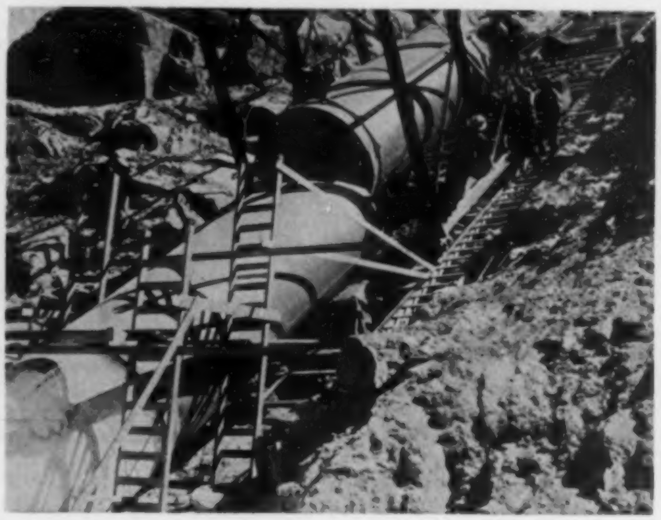
Before being placed in the line, the sections were sand-blasted inside and out, painted on the inside with one coat of coal tar primer, and on the outside with one of synthetic red lead. After a 24-hr interval, the inside was covered with one coat of hot coal-tar enamel. After the field welding was completed, all burns and abrasions were patched, and the outside of the pipe was given a coat of white enamel. An outside finishing coat of synthetic aluminum paint is now being applied.

WELDED JOINTS

Two types of circumferential joints were used for welding in the field. Pipe sections which ranged in thickness from $^3/_8$ to $^5/_8$ in. were made with plain ends, and afterwards a "bell" was rolled on one end of each section so as to overlap the adjoining plain end a distance of $1^1/_2$ in. when installed in the line. All pipe sections having a plate thickness greater than $^5/_8$ in. were beveled for field welding, and a semicircular butt strap was shop-welded to each end of the pipe section. In all cases these straps are 8 in. wide and of the same plate thickness as the pipe section to which they are attached.

Each tangent was provided with a closing sleeve, the inside diameter of which equaled the outside diameter of the standard pipe. The sleeve was made to overlap the adjacent pipe section approximately 6 in. on each end. The purpose of these sleeves was to facilitate the closing of the tangents and to compensate for any small accumulative errors in pipe lengths.

A Venturi tube equipped with two automatic recording meters was installed in the pipe line about 400 ft west of the tunnel portal. The tube, which has an overall length of 35.5 ft, was designed to measure flows in either direction. The throat section is 4.75 ft long and has an inside diameter of about 5 ft. The cone sections are 15 ft long, and each has the same slope from the



STEEL CARRIAGE DELIVERING A PIPE SECTION ON A STEEP SLOPE

throat section to its junction with the 94-in. pipe. The tube was made in the shops of the Department of Water and Power. The housing for the metering equipment is of reinforced concrete construction. In the walls, roof, and floor the slab thicknesses have been made sufficient to prevent movement in that section of the pipe line.

Three expansion joints were placed in the line. One was installed on the east side, and two on the west side of the San Francisquito Canyon. Throughout the remainder of the line an anchor block was placed at each bend so that temperature changes would produce only direct longitudinal stresses.

ANCHORS AND PIERS AT FREQUENT INTERVALS

Since the anchors were designed to restrain movement due to temperature variations from 10 to 120 F, the closing welds on each tangent were made between temperatures of 55 and 60 F. Under operating conditions, when the pipe is filled with water, the temperature of the shell will probably never exceed 80 F. The combined thrust caused by hydrostatic pressure, a temperature rise of 20 or 25 F, and the tendency to shorten longitudinally due to hoop tension, will be less than that produced by a temperature rise of 65 F when the pipe is empty.

Except near the expansion joints in the San Francisquito Canyon, the anchorage at each bend consists of a cage of reinforcing bars bent to conform to the upper half of the pipe elbow and grouted into the rock below. These bars were embedded in concrete which was given a minimum thickness of 12 in. over the top of the pipe and was formed on the rock into which the reinforcing bars were grouted.

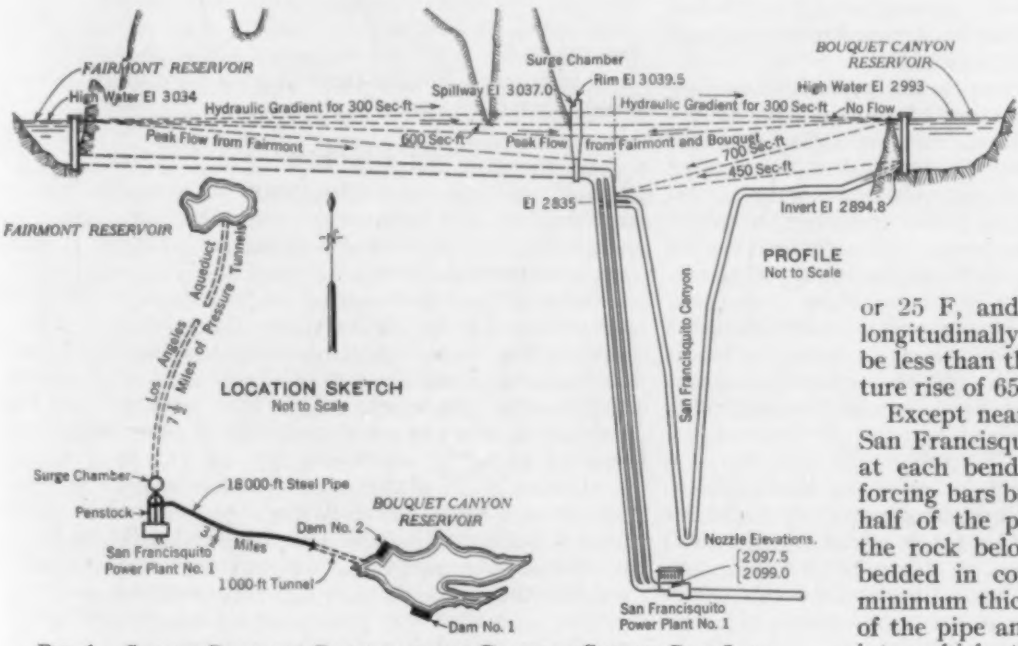


FIG. 1. SKETCH PLAN AND PROFILE OF THE BOUQUET CANYON PIPE LINE
A $3^1/_2$-Mile Conduit 80 to 94 In. in Diameter

Below the expansion joints on the sides of San Francisquito Canyon the vertical bends were anchored by means of steel saddle plates placed over the pipe elbows and welded to reinforcing bars, which were grouted into firm rock. There are five anchors of this type, all

SHOP-WELDING STRAINS RELIEVED IN OVEN HEATED TO 1,200 F

designed to hold the bends against full hydrostatic thrust, but at the same time to allow for some longitudinal movement in the tangents. In addition, an anchor was placed near the middle of each of the five tangents that exceed 500 ft in length, to prevent the pipe from being forced out of alignment by "column action."

Through most of its length the pipe is supported on concrete piers, one pier under each section, to facilitate the laying of the pipe. The maximum spacing of these piers is 27 ft. Under the 30-ft sections, the spacing was made 24 ft, since the thinnest plate was fabricated into the longest sections.

Because of the irregularity of the terrain along the pipe line, an investigation was conducted to determine the best method of supporting the pipe. An estimate was made of the cost per foot of pipe of three types of support: plain concrete piers, reinforced concrete bents, and structural steel bents. The advantageous height for the use of each type was determined. Concrete piers were found to be economical up to an unsupported height of 5 ft; reinforced concrete bents, from 5 ft to 10 ft; and steel bents, over 10 ft.

Provision was made to empty the pipe by installing a blow-off valve at the bottom of each of the 13 depressions in the line. A 12-in. valve was placed in the bottom of San Francisquito Canyon and an 8-in. valve at each of the other positions. On the summits, adjacent to the blow-off valves, 15 air and vacuum valves were installed. On the west summit, adjacent to the San Francisquito Canyon, there are two 8-in. air valves, and on the summit east of the canyon there is one 6-in. and one 8-in. air valve. At each of 11 other summits there is one 6-in. valve. These air valves are sufficient to prevent an appreciable amount of vacuum from forming in the pipe while it is being emptied through the blow-off valves and to prevent trapped air at summits.

Screw-plug pass holes 3 in. in diameter were installed in the pipe shell, about 400 ft apart, to serve as convenient inlets for electric light and power wires and for other small equipment both for construction purposes and for future maintenance. Thirty 15 by 18-in. manholes were placed in the pipe line at intervals determined by the pipe slopes and the requirements of access to the interior for maintenance purposes.

Piezometer connections are being installed that should make it possible to calculate the total loss of head caused by the 84 bends. This will be accomplished by making head-loss measurements by pressure manometer in 2,000 ft of approximately straight pipe, 500 ft of straight pipe, and 500 ft of pipe containing four bends of approximately 30 deg. Provision is also made to measure, by mercury manometers, the hydraulic gradient of the flowing water at each end of the 18,000 ft of pipe and at three intermediate points. In addition, pressure rings have been installed on the 1,000-ft inlet-outlet tunnel for the determination of the flow coefficient.

Laying of the pipe was started in September 1933. It was necessary to build a construction road and a power line paralleling the line. Excavation for the pipe trench, anchor blocks, and piers amounted to 83,000 cu yd. Since some of the steeper inclines exceeded 45 deg, special methods of construction were adopted. Excavation was largely done with bulldozers and scarifiers. On the steeper slopes a tractor equipped with a hoist was stationed at the top of the grade to act as a holdback on the tractor operating the bulldozer or scarifier. The formation was mostly decomposed granite requiring little blasting.

Following excavation, a track having a gage of 9 ft 10 in. was laid on wooden ties and centered on the line of the pipe. On this track, a hoist at the top of the slope lowered the concrete for the piers, which were cast to grade before the pipe was laid.

A steel carriage was built to transport the pipe sections on the track to their final position in the line. The sections were delivered by truck and trailer to the top of the slope, unloaded with a caterpillar crane, and set across two piers. The carriage was then pulled over the pipe section, which was picked up by two 10,000-lb air hoists suspended from the top of the carriage. The pipe, securely anchored in the carriage, was then lowered by an electric hoist to its position in the line. The air hoists were very effective in assembling the pipe in place. By this method, over 300 ft of pipe could be placed by one crew in an 8-hr day.

Pipe sections were delivered by the contractor to a central yard in San Francisquito Canyon about 200 yd below the point where the pipe line crosses that canyon. The pipe was unloaded by crane and immediately put through the sand-blasting plant. All straight sections were sand-blasted automatically; two machines had been installed with the necessary elevators and bins for that purpose. All the tapered sections and elbows were sand-blasted by hand.

FIELD WELDING PROCEDURE

Field welding was done with 13 portable-type arc welders, of 400-amp, 40-v capacity. A power line paralleling the pipe furnished current at 2,200 v, which was transformed down to 440 v.

Field welding was carried on in two 8-hr shifts, and assembling in one shift of the same length. The 27 welders that were required to complete the field welding were selected from a group of 85 who made test plates to qualify for the work. This test required the butt-welding of two 1-in. steel plates 26 in. long, which were beveled to a "V" of 60 deg by the use of flux-coated electrodes. The plates were placed in an overhead position at an inclination of 30 deg, the weld being approximately horizontal. This was the most difficult welding position in the pipe line, as it was necessary to fuse the weld to the upper plate in a direct overhead position.

The pipe that was joined with butt straps was first welded by completing the 60-deg V-weld from the inside. As an additional precaution, the butt straps were welded

to the plate on the outside with a fillet weld of about two-thirds the plate thickness. Joints of the bell-and-spigot type were welded on the inside and outside with a fillet of plate thickness. Approximately 12 passes were required to complete the butt-welding of the 1-in. plate and four passes to complete the fillet welds required by the $^3/_8$-in. plate.

At frequent intervals, strain-gage readings were taken to determine the variation in longitudinal stress in the steel plate adjacent to the welds caused by the welding. This stress was found to be less than 10,000 lb per sq in. in all cases.

FABRICATION OF THE STEEL PIPE

In the fabrication of the pipe by the contractor, the edges of the plates were beveled to provide a V-opening of 70 deg toward the outside of the pipe for machine welding, and one of 60 deg for hand welding. The plates were then rolled to diameter, each plate completing one-third of the pipe circumference. The pipe section was formed by tacking the plates together with hand welds at the bottom of the "V." From one to four passes with the automatic machine were required, depending on the plate thickness. Bare-wire electrodes were used in the machines, and a powdered flux was placed in the "V" to shield the arc and metal from oxygen and nitrogen while welding. The flux was melted into a slag about $^1/_8$ in. thick, which to some extent protected the surface of the weld as it was cooling.

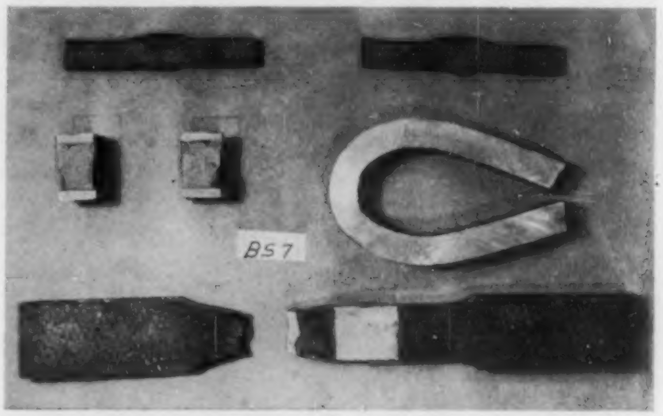
After a pass with the automatic machine was completed, the slag was thoroughly cleaned from the groove and the weld was carefully inspected for flaws by an inspector of the city's Department of Water and Power. If no flaws requiring chipping and rewelding were found, the entire surface of the weld was thoroughly peened and the next pass of weld metal applied. The metal deposited by the last pass was not peened, as this treatment would be likely to prevent leakage through any cracks or other defects during the hydrostatic tests. The weld metal deposited near the ends of the weld was chipped out and the weld completed by hand.

No X-ray photographs were taken of any of the welds. Reliance was placed on a visual examination of the welding process, step by step, by inspectors who were thoroughly familiar with the process and could recognize and eliminate any defects that appeared before they were covered by succeeding passes.

Fifteen test specimens were cut from the finished pipe and 64 others were secured by attaching metal plates to the ends of pipe sections. These plates were made in the presence of inspectors, who determined that the rate of electrode travel, the amperage, the depth of flux, and all other factors governing the quality of the welds were the same as those used throughout the work. The results obtained from specimens cut from the pipe itself were found to be the same as those secured by attaching plates to the ends of pipe sections. The test specimens were of sufficient size to permit tensile, cold-bend, and nick-break tests. A photograph shows the tested specimens from a machine weld on steel plate $^3/_4$ in. thick. Considerable exploration of the weld metal was made by chipping, and excellent information about the quality of this metal was thus obtained, particularly regarding brittleness or hardness that might be caused by variations in flux or machine operation.

Pipe sections that were to be butt-welded in the field were so assembled that the variations from square ends would be compensating. The finished pipe sections could be rotated to three positions or turned end for end to accomplish this.

All sections of the pipe, including the fabricated bends, were given a hydrostatic test. First the pipe section was filled with water to a pressure causing a circumferential stress of 20,000 lb per sq in. in the metal, and each longitudinal seam was inspected. Next, the pressure



Test Specimens Cut from a Machine-Welded Pipe Section Made of $^3/_4$-In. Plate

was reduced sufficiently to cause a stress of 12,000 lb per sq in., and the pipe section was hammered at 12-in. intervals on both sides of each seam. The pressure was then increased to create a circumferential stress of 22,000 lb per sq in. and was held constant for sufficient time to permit close inspection of all three longitudinal seams for any leakage or evidence of high stress at any point.

Only two leaks were revealed by these tests. One occurred at a point where a test specimen had been taken and the other was where part of a machine weld had been chipped out and replaced by hand. It is thus seen that there were no leaks through the machine welding, which aggregated a total length of 51,030 ft. The weld metal adjacent to the two leaks was chipped away and replaced. A second hydrostatic test disclosed no leakage.

Eighty-three bends were required, varying from 3° 44′ to 47° 54′ of curvature. All were made to the outside radius of 23 ft, with a maximum deflection at one weld of 15 deg. Hand-welding with coated electrodes was employed on all parts of these bends. From three to seven passes were used, depending on the thickness of plate. Each weld pass was inspected and peened by methods similar to those used on the machine welds. Any defective part of a weld was immediately chipped and rewelded. The amount of peening required was determined by noting any tendency of the pipe to deform in diameter at any part of the bend. Two bends of approximately 45 deg were fabricated, requiring seven circumferential welds. When completed, these bends varied 3 and 10 min of angle, respectively, from the planned angle, and the ends were within $^1/_8$ in. of the true diameter after the welding was completed. Any large stresses set up by welding would have distorted the ends considerably.

WELDERS QUALIFIED BY TESTS

All welders fabricating bends were required to make "coupons" that passed a modified requirement of the American Society of Mechanical Engineers Class No. 2 Welding Code, using the specified test values but fabricating a "coupon" of sufficient size to mill out one sample each for full-section tension, cold-bend, and nick-break tests. The bends were rotated with cranes so that all this welding was done on the upper half of the

pipe circumference, each pass being completed for the entire circumference before the next was started.

Boiler heads were welded to the ends of all bend sections, and the same hydrostatic test was applied that was used for the straight pipe sections. No leaks developed except at the temporary welds attaching the boiler heads to the bends. It is interesting to note that these temporary welds were only completed for about one-half the plate thickness. Butt-welding was used

INTERIOR OF A 91-IN. PIPE AFTER WELDING
Enamel Applied Before Installation, Except Adjacent to Welds

entirely for this purpose. The heads were butt-welded to the pipe sections made from $\frac{3}{8}$-in. plate by the use of square plate ends and a single pass with $\frac{3}{16}$-in. to $\frac{1}{4}$-in. electrodes. The test pressures produced a longitudinal stress in the plate adjacent to these welds of 11,000 lb per sq in. (half of the 22,000 lb per sq in. circumferential stress) and about twice as much stress across the welds, that is, at least 22,000 lb per sq in. This is a practical demonstration of the factor of safety that can be obtained by butt-welding large pipe. At least $2\frac{1}{2}$ times the thickness of weld metal in these temporary welds was used to complete the finished bends.

All the bends and the straight sections of pipe that were made from plate of $\frac{5}{8}$ in. thickness or more were heated to a temperature of 1,200 F for one hour per inch of plate thickness to relieve internal stresses. The oven in which this work was done was fitted with 16 electric pyrometer indicators that could be placed against the pipe while it was in the oven. The period of heating for relief of stress was considered to commence when all these indicators had reached a temperature of 1,200 F. The method of supporting the pipe at this temperature is shown in a photograph. The temperature used was that at which a slight indentation of the plates was produced at the four points of support.

The value of peening each pass of an electric weld was illustrated in the fabrication of the circular I-beam ring girders, which were built up with two rolled plates for flanges and with circular cut segments for the web. In the first ring, which was fabricated without peening, the weld and the flange ends were drawn toward each other from $\frac{1}{8}$ to $\frac{1}{4}$ in. Controlled peening of each of three passes of the weld eliminated this difficulty.

PIPE COATINGS

During the construction period it was desired to keep the temperature of the pipe as low as possible for several reasons; more comfortable conditions inside the pipe for welders; less movement of the pipe and less stress on partly completed welds; and a smaller difference in temperature between the top and bottom plates during welding. Samples of all available exterior pipe coatings were tested for heat-absorbing qualities (with the results given in Table I) and also for durability. The temperature of metal covered with synthetic red paint was found to be surprisingly low. Because of superior durability, this was selected for the priming coat. For the final coat on the pipe an aluminum paint with phenol-resin base was chosen.

TABLE I. MAXIMUM TEMPERATURES, FAHRENHEIT, OF $\frac{3}{8}$-IN. STEEL TEST PLATES WHEN COATED WITH VARIOUS COLORED PAINTS AND EXPOSED TO THE SUN

Coating	Temperature of Plate	Increase Above Shade Temperature	Coating	Temperature of Plate	Increase Above Shade Temperature
Whitewash	105	7	Green, 2 kinds	133–136	35–38
White paint, 3 kinds	109–114	11–16	Dark red primer	133	35
			Gray primer	135	37
Red lead primer, synthetic	115	17	Steel plate, sand-blasted	136	38
Aluminum, 4 kinds	121–133	23–35	Brown primer	138	40
Grey enamel	124	26	Black paint, 3 kinds	138–141	40–43

Coating material for the inside of the pipe was selected after careful tests, and its application was closely checked. One of the most important tests utilized an electric spark to locate included gas bubbles or other defects in application. A brush with a metal frame and fine wire bristles was connected to the output of a Ford spark coil and passed over the completed enamel. Where the coating was less than $\frac{1}{32}$-in. thick, the spark penetrated it and continued to penetrate as the brush was moved until it reached a length of about $\frac{3}{8}$ in. This spark was clearly visible and audible. No difficulty was found in observing the penetrations. These points were marked and retested after more enamel had been applied. A maximum of 70 penetrations was found in one 20-ft length of pipe. When the enamelers became more proficient, the average number of penetrations was 10 for a 30-ft section of pipe. The testing procedure was repeated after the priming and enameling of the welds had been completed in the installed pipe. This method of inspecting coal-tar enamels is faster and more efficient than any other and has been found to give excellent results.

FIELD SURVEYS

In this project the various surveys were controlled by a triangulation net connected with a recently completed net of the U. S. Coast and Geodetic Survey and by a precise level circuit that joined the system of precise levels of the original Los Angeles Aqueduct. The location of the pipe line was selected after cost studies were made on three surveyed routes. Preliminary locations were made on a scale of 1 in. to 500 ft, and the final location was made on a topographic map plotted by plane table to a scale of 1 in. to 100 ft and controlled by a profiled preliminary traverse with frequent bench marks.

A considerable effort was made to obtain an economic balance between the cost of excavation and the height of the required concrete piers, with the result that there was somewhat more excavation than is usual on pipe lines of this type. The pipe line was completed by March 28, 1934, on which day storing of water in the Bouquet Canyon Reservoir was begun.

OUR READERS SAY—

In Comment on Papers, Society Affairs, and Related Professional Interests

Closure on Method of Computing Beam Deflections

DEAR SIR: In answer to Professor Grinter's discussion, in the April issue, on my article on an improved method of computing beam deflections, in the February number, the following statements appear to be in order.

The method discussed is analytical rather than semigraphical, because none of the quantities are scaled from drawings and the figures used by the computer can be drawn free-hand. The constants, which must be memorized or developed, are the same as the constants used in the moment-area method: namely, the distances from the ends of members to the centers of gravity of the moment diagrams and the areas of the moment diagrams. Nothing more has to be remembered or derived than in the case of the moment-area method.

The advantages of the method are its pictorial clearness, which Professor Grinter has mentioned, and its abbreviated notation. Both of these qualities facilitate the use of the method in the solution of advanced problems in continuous beams and frames.

R. W. STEWART, M. Am. Soc. C.E.
Engineer, Bridge and Structural Design

Los Angeles, Calif.
April 30, 1934

A Record of Tree-Ring Growth from Idaho

TO THE EDITOR: In connection with the subject of deficient run-off of streams in numerous Western states during recent years, the accompanying Fig. 1 may be of interest. This diagram shows tree-ring widths of the oldest tree ever found in southern Idaho. Although the growth of trees may be affected by factors other than precipitation, yet that is doubtless the most important.

This record discloses that there have been several periods during the 1,600 years of the life of the tree when its rate of growth, and presumably precipitation, were considerably less than during the relatively recent period that the region has been settled. The tree grew on the Snake River plains at a point where ground water is 200 ft below the surface. Thus it received no supply of moisture except from precipitation. At Idaho Falls, Idaho, the nearest Weather Bureau station, the average annual precipitation is 12.3 in.

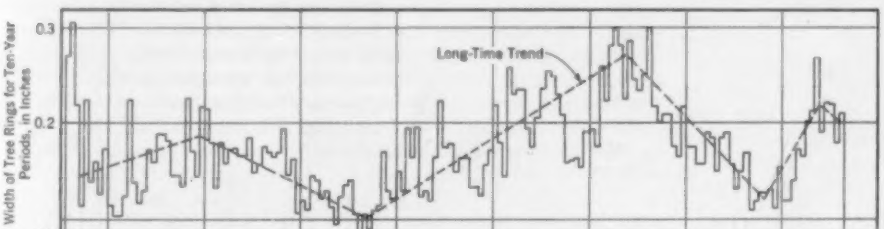


FIG. 1. WIDTH OF RINGS OF A DESERT JUNIPER NEAR IDAHO FALLS, IDAHO
Record for 1,600 Years Shown by 10-Year Averages

LYNN CRANDALL, M. Am. Soc. C.E.
District Engineer, U. S. Geological Survey

Idaho Falls, Idaho
April 26, 1934

Three-Hinged Arch Bridges

TO THE EDITOR: The April issue of CIVIL ENGINEERING has an excellent article on "Temporary Hinges in an Arched Bridge," by Leon Blog. In this article Mr. Blog describes the Fourth Street Viaduct, built in 1931 across the Los Angeles River, and states that, "So far as is known, this was the first reinforced concrete arch bridge to be erected by the three-hinged method in the United States."

A bridge across the Rogue River in Oregon was designed in 1929 by C. B. McCullough, M. Am. Soc. C.E., Bridge Engineer of the Oregon State Highway Department, and erected in 1930 by the Oregon State Highway commission. Plans for it are shown on pages 332 and 333 of *Elastic Arch Bridges* by C. B. McCullough and E. S. Thayer, which was published in 1931 by John Wiley and Sons. This structure incorporates the use of the Consideré hinges, which are identical in design with the hinges used for the Fourth Street Viaduct in Los Angeles.

Mr. McCullough has utilized the same design for the Wilson River Bridge, described in the *Engineering News-Record* of August 27, 1931, and for the Ten-Mile Bridge and the Big Creek Bridge on the Oregon Coast Highway. He also used these hinges in the design of the Cape Creek Bridge in 1931, and of the Santiam Bridge in 1932.

This same detail is incorporated in the design of 33 concrete arches of the Alsea Bridge, the Yaquina Bridge, the Coos Bay Bridge, the Umpqua Bridge, and the Siuslaw Bridge, which will soon be constructed on the Oregon Coast Highway. These arches have spans ranging from 151 to 270 ft in length. It appears then that this type of construction is no longer new in this country.

M. E. REED, M. Am. Soc. C.E.

Portland, Ore.
April 25, 1934

The Engineer and Public Utilities

TO THE EDITOR: The article by Mr. McDonald, in the March issue, comes at a time when many are perplexed by the issues raised by increasing interest in the planning and construction of municipal light and power plants. Surely it is not logical to have a municipal project for furnishing light and power unless rates are high enough to meet all fixed charges and operating costs, including the important but frequently overlooked item of taxes.

If it is not feasible to levy general property taxes, sales taxes, and other proper taxes on such a municipally owned project, there should be devised some method of turning over each year to the general funds of the local government a proper allowance for taxes. In preliminary reports on municipal plants an allowance should always be made for taxes, which will be lost by displacing or reducing in value the taxable property of an existing privately owned utility. An engineer who makes such reports without introducing the subject of taxes is ignoring a fundamental element in the problem.

Since there is a distinct trend toward public ownership of the power utilities, engineers engaged in any branch of such work have the obvious duty of appraising fairly the relative serviceability and economy of private and public enterprises in this field. Lower rates are not less costly if the quality of the service is inferior. Recent experience has illustrated the sad result of competition between publicly owned and privately owned utilities in a particular locality. Municipal plants have been most successful where they

have had the field to themselves. The progressive State of Wisconsin will not permit ruinous competition between public and private ownership. There, purchase or condemnation of existing facilities is mandatory for municipal ownership.

No city council, village board, or county board of supervisors should come to a decision to build a local light and power plant or to build or acquire a distribution system without a reliable engineering report based on painstaking surveys and investigation. Such a report should contain accurate estimates of construction costs, operating costs, including adequate repairs and maintenance and taxes, and fair estimates of fixed charges and probable revenue.

Where rate adjustment is possible, new construction and municipal operation should not be undertaken merely to satisfy political demands. First, the possibilities of satisfactory rate reductions should be determined. The processes of national recovery are not aided by the needless destruction of existing investments, except possibly in the case of a very few of the so-called "yardstick installations."

According to Mr. McDonald, the Federal Power Authority should absorb, for an interim, losses caused by installing a promotional scale of lower rates calculated to increase the demand for light and power. Ordinarily, as Mr. McDonald points out, the lower the rate, the greater the consumption of power. However, such considerations suggest the advisability of trial periods of rate adjustment with marked reductions in the initial brackets of the rate schedules to promote increased use of light and power, with consequent reduction in the average cost of these utilities.

Mr. McDonald has helped to clear away much of the confusion that has existed in the consideration of governmental ownership in all types of revenue-producing activities. Before embarking on his large paternalistic scheme of interim subsidy for operating power companies, let us first determine the possibilities of adjusted rate schedules and improved local facilities for furnishing light and power to our people in a truly democratic way, with emphasis on the "survival of individual freedom, of initiative, and of enterprise."

ARTHUR WARDEL CONSOER, M. AM. SOC. C.E.
Vice-President and General Manager, Consoer, Townsend, Older and Quinlan, Inc.

Chicago, Ill.
April 20, 1934

Our Paternalistic Government

TO THE EDITOR: In "An Equitable Theory of Government Ownership and Operation," in the March issue, Mr. McDonald complains that government is restricting the progress of business and that it is collecting taxes to pay the costs of government. A year ago business was begging the government to rescue our economic system; now it resists "meddling." I shall show some inconsistencies in this attitude.

Under the NRA codes, the government has handed over to business control of its own activities, and permitted untimely price-raising. The *New York Times* of March 11 stated that department store sales showed a gain of 16 per cent in February 1934 over the sales for February 1933, but this was due to a 25 per cent advance in prices. This means that less merchandise was sold than in the demoralized Feburary 1933.

The true functions of government, says Mr. McDonald, are to provide water supply, sanitation, police, fire, and school services. Why stop here? Add group medical service, electric power, community hospitalization, communications, higher education, and playground facilities. Then "picture a community congested, concentrated, throbbing with business and social life..." as Mr. McDonald advises. These should not be revenue-producing activities. The financial loss is met by the community but is recouped socially and economically. The benefit is general, since the community must have schools, hospitals, and electricity.

If each consumer is to pay for specific services, the government must revise school taxes, lower first-class postal rates and raise second and third-class rates, collect for fire calls, reduce residential electric rates and raise power rates, and charge to the respective owners the cost of intervention in foreign lands to protect private property. Business itself does not make equitable charges for services. Why should it expect the government to do so?

All students of politics are not agreed that the regulation of utilities by state commissions is as complete or effective as Mr. McDonald claims it is. The Government should be allowed to carry out its Tennessee Valley Authority program unhampered, and complete accounting systems should be set up to determine the costs by subdivisions. Simplicity should be the aim, with holding companies and management charges ruled out.

After discussing "the destructive effect [of the Tennessee Valley Authority program] on existing private enterprise," Mr. McDonald suggests the electrification of railroads without discussing the effect on the coal industry. Would there be a complaint similar to Mr. McDonald's? Would the output be cut 30 per cent with even greater depreciation of capital and loss to wage-earners?

A state mutual power company would not be satisfactory as a yardstick because the financial structure is too circuitous. In fact public ownership with private control and management would constitute a subsidy, to which Mr. McDonald objects, and an "endeavor blazoned for the benefit of the few."

In conclusion, a quicker and wider economic and social betterment is not in general compatible with mandatorially self-supporting rate structures. A subsidy to business does not necessarily accelerate general-welfare benefits. If anyone is to be subsidized, why should it not be the consumer?

S. B. FOLK, ASSOC. M. AM. SOC. C.E.
Assistant Professor of Mechanics
Ohio State University

Columbus, Ohio
May 1, 1934

Slabs on Non-Rigid Supports

TO THE EDITOR: In the résumé of his report on "Concentrated Loads on Slabs," in the March issue, Professor Morris shows conclusions that give a safe, economical, and practical means of designing slabs rigidly supported on two edges. It is evident, however, that there is still room for investigation of slabs having non-rigid supports and of slabs continuous over two or more spans.

Since Professor Morris's report was completed, I have derived an equation for the effective width of a slab on flexible girder supports. Only a centrally applied load was considered. A brief explanation of the method used and the results obtained is presented here.

Assume any single-span slab supported on two edges, in which s equals the span of the slab and b equals the width of the slab, or the span of the girders. With a given load, P, concentrated at the center, the girders will deflect some definite amount, D_O, with respect to their supports. At the same time the slab, at P, will deflect some definite amount, d_O, with respect to the girders at the center of the girder span. Combining the ratio D_O/d_O with the assumption that the deflection of a slab varies as the cube of the span, an equation can be derived that gives the magnitude of the slab reaction at any point along the girder in terms of r_O, the maximum value of the slab reaction opposite the load.

From symmetry the total slab reaction on one girder is $P/2$, which is equal to $r_O E$, where E is the effective width. Then by integrating the equation for the slab reaction between the limits of the ends of the girder and equating this result to $r_O E$, the r_O terms cancel and the equation becomes

$$\frac{E}{s} = \left[\frac{\theta}{2} + \frac{1}{4}\sin 2\theta\right] + \frac{D_O}{d_O}\frac{s^2}{b^2}\left[\frac{\theta}{2} - \frac{1}{4}\sin 2\theta\right]$$

in which θ is the angle in radians between a line through P, perpendicular to the girders, and a line connecting P with the end of a girder.

If the supports are made rigid, D_O equals zero and the equation reduces to $\frac{E}{s} = \left[\frac{\theta}{2} + \frac{1}{4}\sin 2\theta\right]$. This is Equation 3 in Professor Morris's report published by the Engineering Experiment Station of Ohio State University, in *Bulletin No. 80*.

To illustrate the effect of the girder deflection on the distribution of the load, assume that a slab has a span, s, equal to 8 ft; a width of b equal to 28 ft; and that the deflection, d_O, of the slab between the girders at P is 0.12 in. Then if D_O equals 1.20 in., E equals 9.60 ft. Also, if D_O equals 0.30 in., E equals 7.08 ft. If the slab supports are rigid, D_O equals zero and E equals 6.24 ft. In practice, values of E should be increased to allow for the width c, over which the load is applied. If this width is taken as 15 in. with rigid supports, E becomes 7.49 ft. By the formula given in the 1931 specifications of the American Association of State Highway Officials, E

$= 0.7(2x + c) = 6.48$ ft, where x is the distance from the wheel to the nearest support. By Equation 1, given in Professor Morris's report, $E = 0.6s + 2c = 7.30$ ft.

G. T. Parkin, Jun. Am. Soc. C.E.
Designer, North Carolina State Highway and Public Works Commission

Raleigh, N.C.
April 26, 1934

Municipal Electric Plants

Dear Sir: In an article entitled "The Legality of a Self-Supporting Rate Base," in the March issue, James P. Gifford writes: "What is the possibility of obtaining from a court a mandatory requirement that governmentally owned and operated (electric) services should have a rate structure providing a self-supporting revenue basis? Speaking of the existing legal system, the answer is simply this: There is no such possibility."

He may be right in his assumption that any state or Federal power project, in the absence of constitutional amendment to the contrary, is beyond control; but when he includes municipal undertakings, he is not. The sovereign power that authorizes a municipality to engage in the electrical business can prescribe conditions for its conduct and can provide penalties for violation of those conditions. For over forty years the municipal electric law of Massachusetts has provided that the manager of every municipal plant must set his rates high enough to earn: all operating expenses; interest on the outstanding debt; a fund for retiring the debt within 20 years; a depreciation allowance fixed by law; and any deficit in the operation of the preceding year.

The managers obey this law: the taxpayers see to that. Not only must any deficit be recovered in the rates of the ensuing year, but the law also provides that the amount of the deficit must be included in the tax levy—presumably to keep the plant solvent until the loss can be made good out of income. Although the taxpayer might hope to be reimbursed eventually, he is notoriously reluctant to part with his money and too influential to have temporary financing foisted on him. So he is not called on, and the plants invariably pay their own way, and even more.

The consolidated balance sheet of the municipal electric plants of Massachusetts shows that only about 10 per cent of the cost of the properties is represented by outstanding debt. The rest has been financed from surplus earnings. Unfortunately that is not the whole story.

The influence of the taxpayer is so strong that he can exact a subsidy from the customers of the electric plant. Over half the plants in the state are not only earning enough to comply with the requirements of the law and to finance additions to their properties, but also are paying surplus earnings to the municipalities in order to reduce the tax rate. One of them is devoting nearly 30 per cent of its gross income to that purpose, although this is equivalent to an overcharge of 3 cents in the highest block in a stepped rate.

Surplus earnings are limited by law to 8 per cent of the cost of the plant, but this limit has been exceeded with impunity. Observance of this limit can be compelled if 20 taxpayers or the department of public utilities petition the superior court to order the manager to comply with the law, but I do not recall a single instance of such action. So long as rates do not increase, the public is indifferent. Consumers in one town may be paying an average of $2^1/_2$ cents for 600 units a month; whereas in an adjoining town, where conditions are so favorable that the average could be $1^3/_4$ cents, the consumers are paying nearly 4 cents, year after year, without protest.

Experience in Masschusetts clearly demonstrates that a self-supporting rate may be required, but it also indicates that it is the consumers rather than the taxpayers who need protection against operation for political ends. This experience also raises interesting speculation as to the fate of those dependent on state and Federal power projects if the plants should fall into the hands of an administration committed to "less government in business!"

Earl H. Barber, Assoc. M. Am. Soc. C.E.

Reading, Mass.
April 24, 1934

Variation of Curve Formula

Dear Sir: In connection with Mr. MacDowell's article, "Passing a Curve Through a Fixed Point," in the January number of Civil Engineering, I would like to submit the following solution, which is adaptable to logarithmic computation and is based on the fact that the length, BC, equals $AB \cos \frac{I}{2}$ (Fig. 1). It is obvious that this relation might also be utilized in laying out a short curve

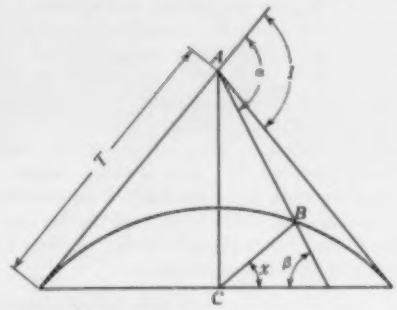


Fig. 1. Method of Passing a Curve Through a Given Point, B

with the use of tapes alone, after the points of tangency and the point C are located. By reference to Fig. 1,

$$\beta = \alpha - \frac{I}{2} \qquad [1]$$

$$x = \cos^{-1}\left(\frac{\cos \beta}{\cos \frac{I}{2}}\right) \qquad [2]$$

$$T = \frac{AB \sin (\beta + x)}{\cos \beta \tan \frac{I}{2}} \qquad [3]$$

George H. Dell, Assoc. M. Am. Soc. C.E.
Associate in Civil Engineering
University of Illinois

Urbana, Ill.
April 27, 1934

Sodium Sulfate in Soundness Tests

To the Editor: The article on "Soundness Tests for Sewage Filter Media," by Professor Payrow, in the February issue, is a timely addition to much-needed information in this field. In this article he states, "It is obvious that a variation in temperature may affect the results of the test considerably." A few comments based on my own observations may assist in amplifying and clarifying this conclusion.

As shown in Professor Payrow's Fig. 1, sodium sulfate has a peculiar solubility curve and is known to crystallize in three forms: the decahydrate, $Na_2SO_4 10H_2O$; the heptahydrate, $Na_2SO_4 7H_2O$; and the anhydrous, Na_2SO_4. There is general agreement that the decahydrate changes into the anhydrous form between 32 and 33 C. Apparently, also, the heptahydrate is formed alone below 12 C; whereas both the heptahydrate and the decahydrate may exist together between 12 and 24 C. The heptahydrate is unstable and may pass entirely into the decahydrate in solution. If the decahydrate alone exists between 24 and 32 C, then the temperature of 25 C suggested by Professor Payrow would tend to produce decahydrate crystals only. Since these occupy the greatest space, they are presumably the most destructive.

However, I am inclined to believe that at lower temperatures the disrupting effect, if any, will be more pronounced and that it is caused by the formation of larger crystals rather than by the particular form of the crystals. In support of this theory is the fact that the disruptive effect on trickling filter media, in actual service, is greatest during periods of continued cold weather, and the colder the weather the more pronounced the disintegration.

It would seem, then, that the controlling factor in disintegration is not the form of the ice crystals, but rather their number.

In the test procedure the limits of temperature of the solution during immersion of the specimen should be definitely established, since this appears to be the criterion by which consistent results are most likely to be obtained. Furthermore, the test should be kept as simple as possible and sufficiently practicable for application without elaborate equipment. A temperature of 21 C (70 F) with a tolerance either way of 5 C seems attainable in most laboratories without difficulty.

It is questionable whether the sodium sulfate test is as applicable to slag as to stone, particularly in the case of the more porous slags. The sodium sulfate test is admittedly an accelerated test. In spite of its imperfections it is believed to be reliable for predicting the soundness of proposed filtering material within reasonable limits as to time required to obtain results.

Actual freezing and thawing tests will give no additional information, and the time required for them is much longer. Moreover, when freezing and thawing tests are used, I cannot see the logic of disregarding technique, particularly in such matters as temperature control, in view of what takes place in some trickling filters exposed to wide variations in winter temperatures.

FRANK WOODBURY JONES
Sanitary Chemist
George B. Gascoigne

Cleveland, Ohio
May 4, 1934

The Engineer and the Contractor

TO THE EDITOR: The article by Herbert S. Crocker, in the March issue, merits thoughtful study by members of the profession. If the circumstances that prompted Colonel Crocker's discussion did not exist, many construction undertakings would represent greater service value, less capital outlay, and a more genuine reason for pride of accomplishment on the part of the engineer. What engineer can justly feel satisfied with completed work that represents an excessive expenditure of funds and a silently sustained loss on the part of the contractor? These are the two inevitable results of either inexperienced field engineering or failure on the part of responsible authority to properly recognize a conflict between contract stipulations and unanticipated developments in the field.

In calling attention to one-sided contract stipulations, Colonel Crocker refers to the familiar phrase, "to the satisfaction of the engineer," often used in contracts, as "a sort of blanket clean-up clause." This is a very appropriate definition, since the sole purpose of that clause is to protect the engineer if, in preparing the contract, he inadvertently fails to impose upon the contractor complete responsibility for any of the numerous contingencies over which the contractor has no control.

In a discussion of the Twelfth Street Trafficway, in Kansas City, published in the 1916 TRANSACTIONS of the Society, I commended the engineers for their pronounced policy of eliminating the usual unreasonable and unfair contract stipulations. My comment pointed out that a contractor could perform work entirely above criticism and yet not be able to comply with the term, "to the satisfaction of the engineer." It is unreasonable enough, for example, to hold a contractor wholly responsible for contingencies and for errors in design and mistakes of engineering judgment, as is attempted in the execution of many construction projects.

Colonel Crocker suggests that the colleges place more emphasis on problems to be encountered in the field. Such a step would be helpful if it did not make too great a demand on the time devoted to mathematics, physics, mechanics and allied subjects, and their application to the science of engineering. The more thorough his preparation in such fundamental subjects, the more readily does the graduate respond to training in the field under competent direction. An effective method of overcoming existing unsatisfactory conditions would be to impress the student with the fact that a responsible contractor is essential to every construction undertaking; that his services should not be subjected to unnecessary hazards; and that the removal of uncertainties and the minimizing of risks not only discourages submission of bids by the irresponsible contractors but also reduces the final cost of a project.

Another excellent point in Colonel Crocker's article is that engineers in responsible positions should constantly strive to overcome any tendency toward impatience with a lack of immediate ability on the part of young graduates to render service of practical value. They should encourage initiative and a sense of responsibility and take every opportunity to point out the close relationship between the principles learned in the graduate's training and the specific problem at hand. The contractor, too, must participate to the extent of calling to the attention of responsible authority any unreasonable demand of the field engineer. The really conscientious field man will encourage such a procedure.

HOWARD W. HOLMES, M. Am. Soc. C.E.

Alameda, Calif.
April 20, 1934

Construction Methods Should Be Learned in the Field

TO THE EDITOR: Colonel Crocker's article on "The Engineering Viewpoint," in the March issue, contains interesting and valuable suggestions for the efficient practice of engineering in the field. However, Colonel Crocker calls upon the colleges to perform a function which, I think, it would be most unwise for them to undertake. After relating instances of the difficulties that inexperienced college graduates encounter when they are given positions of responsibility on construction work, he says, "The solution of the problem lies in the inclusion in college curricula of courses in the technique of construction machinery and its uses, as well as studies of field methods of erection and handling of materials in an economical manner."

No one, I am sure, will contradict me when I assert that the curricula of colleges and technical schools are crowded. The problem is therefore not a question of making an addition to the existing curricula, but of determining what subjects the colleges can teach to best advantage.

Colonel Crocker's division of engineering into theory and practice is acceptable for present purposes. He urges that the colleges teach more of the practice. I hold, on the contrary, that colleges should teach principally the theory of engineering, including only enough construction practice to keep the theory close to reality.

It seems to me that college is the best, if not the only place where the would-be engineer can gain a clear understanding of the fundamental principles of stresses, strains, and the strength of materials. Without such a clear understanding, he will be at a disadvantage throughout his career. These fundamentals change little from year to year. Progress is made by means of gradual additions to the common fund of knowledge, clearer presentations, and better teaching methods. The colleges can, and commonly do give students a fairly complete and adequate comprehension of the theory underlying engineering design, but this is not enough. Their efforts should be devoted to the teaching of theory by constantly improved methods. If this aspect of the engineer's training is well developed in college, it rarely needs attention later.

On the other hand, construction practice varies from year to year, from place to place, and from job to job. I do not assert that the colleges could not keep abreast of the advancing art of construction and that they could not adequately teach construction methods and equipment, but the effort required would be disproportionate to the results. Construction methods can be learned so much better in the field from actual experience that it seems really wasteful for the colleges to devote more time to teaching these methods than is contributory to an understanding of the theory.

As Colonel Crocker points out, the solution of the problem lies in not placing inexperienced men, even though they be college graduates, in responsible charge of construction. Let the colleges teach theory better and not dissipate their energies at tasks they cannot possibly do so well.

JOHN R. NICHOLS, M. Am. Soc. C.E.
Consulting Engineer

Boston, Mass.
May 1, 1934

Sixty-Fourth Annual Convention

Vancouver Hotel, Vancouver, B.C., Canada, July 11–14, 1934
Program of Sessions, Entertainment, and Trips

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VANCOUVER'S WATER FRONT AND SKYLINE

General and Technical Meetings

HELD JOINTLY WITH THE ENGINEERING INSTITUTE OF CANADA

Wednesday—July 11, 1934, Morning

9:00 **Registration**

10:00 **Sixty-Fourth Annual Convention of the Society and Western Professional Meeting of the Engineering Institute of Canada called to order by**

E. A. CLEVELAND, *M. Am. Soc. C.E., Chief Commissioner, Greater Vancouver Water District,* Vancouver, B.C., Canada; *Chairman, Local Committee on Arrangements.*

Welcome to Visiting Engineers
His Worship, Mayor Taylor of Vancouver.

10:10 **Address of Welcome**

F. P. SHEARWOOD, *M. Am. Soc. C.E., Chief Engineer, Dominion Bridge Company, Ltd.,* Montreal, Que., Canada; *President, Engineering Institute of Canada.*

Response and Annual Address

HARRISON P. EDDY, *President, American Society of Civil Engineers; Consulting Engineer,* Boston, Mass.

11:30 **Through British Columbia Waters to Alaska**

Commander B. L. JOHNSON, *R.N.R., D.S.D., Lloyd's Agent in British Columbia.*

12:20 **Business Meeting of the Society**

12:30 **Luncheon at Hotel Vancouver**

Chairman, P. H. BUCHAN, *Esq., Chairman, Vancouver Branch, Engineering Institute of Canada.*

Following luncheon, there will be an address on "The Growth and Work of the Association of Professional Engineers in British Columbia," by A. S. GENTLES, *Esq., President, The Association of Professional Engineers of British Columbia.*

Tickets for the luncheon are $1.00 each.

Wednesday—July 11, 1934, Afternoon

SYMPOSIUM ON "THE DEVELOPMENT OF THE COLUMBIA RIVER DRAINAGE BASIN"

2:30 (1) **Canadian Aspects of the Columbia River Drainage Basin**

Maj. J. C. MACDONALD, *Esq., Comptroller of Water Rights, Province of British Columbia,* Victoria, B.C., Canada.

3:10 **Discussion opened by**

C. E. WEBB, *Esq., District Chief Engineer, Dominion Water Power and Hydrometric Bureau, Department of the Interior,* Vancouver, B.C.

J. P. FORDE, *Esq., District Engineer, Department of Public Works of Canada,* New Westminster, B.C.

3:30 (2) **Power and Navigation Features of the Columbia River Below the Mouth of the Snake River, with Particular Reference to the Bonneville Power and Navigation Project**

C. F. WILLIAMS, *Esq., Major, Corps of Engineers, U. S. Army; District Engineer, Bonneville Dam Section,* Portland, Ore.

4:05 (3) **Reclamation Features of the Columbia River Drainage Basin, with Particular Reference to the Columbia Basin Project and the Grand Coulee Development**

FRANK A. BANKS, *Assoc. M. Am. Soc. C.E., Construction Engineer, U. S. Bureau of Reclamation,* Almira, Wash.

4:25 **Discussion opened by**

CHARLES B. SMITH, *Esq., Engineer, War Department,* Seattle, Wash.

Wednesday—July 11, 1934, Evening

7:00 **Dinner and Entertainment**

Chairman, HARRISON P. EDDY, *President, American Society of Civil Engineers.*

Following the dinner, there will be an address on "The Work of the Royal Engineer in British Columbia (1858–1865)," by His Honor, Judge F. W. HOWAY, *L.L.B., L.L.D., F.R.S.C.*

Tickets for the dinner and entertainment are $3.25 each.

All Sessions, Trips, and Entertainments, Except the Technical Sessions on Thursday, July 12, Are to Be Held Jointly with the Engineering Institute of Canada.

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MARINE DRIVE, NORTH SHORE, VANCOUVER, B.C.

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LIBRARY OF UNIVERSITY OF BRITISH COLUMBIA, AT VANCOUVER

Sessions of Technical Divisions Occupy Entire Day

Thursday—July 12, 1934, Morning

IRRIGATION DIVISION AND POWER DIVISION

9:00 **Central Valley Project of California**

EDWARD HYATT, *M. Am. Soc. C.E., State Engineer,* Sacramento, Calif.

Discussion

10:00 **Model Studies of Draft Tubes with Particular Reference to Those of the Bonneville Development**

C. A. MOCKMORE, *Professor and Acting Head, Department of Civil Engineering, Oregon State College,* Corvallis, Ore.

Discussion

HIGHWAY DIVISION AND CONSTRUCTION DIVISION

9:00 **Alaska-United States Highway**

MALCOLM ELLIOTT, *M. Am. Soc. C.E., Major, Corps of Engineers, U.S.A.; Office of Chief of Engineers, U.S.A.,* Washington, D.C.

9:30 **Discussion**

10:00 **Inter-American Highway**

E. W. JAMES, *M. Am. Soc. C.E., Chief, Division of Design, Bureau of Public Roads,* Washington, D.C.

10:30 **Discussion**

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BURRARD STREET BRIDGE

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ALBERTA POOL ELEVATOR

© Hileman

ALONG THE GOING-TO-THE-SUN HIGHWAY, GLACIER NATIONAL PARK

MOUNTAIN FROM NEW SUNRISE HIGHWAY, RAINIER NATIONAL PARK

Sessions of Technical Divisions (*Continued*)

Thursday—July 12, 1934, Afternoon

HIGHWAY DIVISION AND CONSTRUCTION DIVISION

2:00 **The Fort Peck Project**

THEODORE WYMAN, JR., *Esq., Captain, Corps of Engineers, U.S.A.; Missouri River Division*, Kansas City, Mo.

2:30 **Discussion**

3:00 **Going-to-the-Sun Highway**

JEAN EWEN and A. V. EMERY, *Esquires, Assistant Highway Engineers, Bureau of Public Roads*, Portland, Ore.

3:30 **Discussion**

IRRIGATION DIVISION AND POWER DIVISION

2:00 **The Silt Problem**

J. C. STEVENS, *M. Am. Soc. C.E., Consulting Hydraulic Engineer*, Portland, Ore.

3:00 **The Bonneville Power House and Generating Equipment**

B. E. TORPEN, *M. Am. Soc. C.E., Engineer, Bonneville Dam*, Portland, Ore.

4:00 **Discussion**

SANITARY ENGINEERING DIVISION

2:00 **Sanitary Engineering Experiences with a Large Public Water Supply Enterprise in the West**

JOSEPH D. DE COSTA, *Assoc. M. Am. Soc. C.E., Sanitary Engineer, East Bay Municipal Utility District*, Oakland, Calif.

2:50 **Experiences in the Treatment of Pacific Coast Water High in Color, Iron, Manganese, and Sulfurated Hydrogen**

JEPTHA A. WADE, *M. Am. Soc. C.E., Chief Engineer, California Water Service Company*, San Francisco, Calif., and KENNETH W. BROWN, *Esq., Sanitary Engineer, California Water Service Company*, Stockton, Calif.

3:40 **Municipal Refuse Problems and Procedures on the Pacific Coast**

CHESTER G. GILLESPIE, *M. Am. Soc. C.E., Chief, Bureau of Sanitary Engineering, State Department of Public Health*, Berkeley, Calif., and EDWARD A. REINKE, *Assoc. M. Am. Soc. C.E., Research Engineer, Bureau of Sanitary Engineering, State Department of Public Health*, Berkeley, Calif.

4:30 **Progress Report of Committee on Salvage of Sewage**

A. W. RAWN, *M. Am. Soc. C.E., Chairman; Assistant Chief Engineer, Los Angeles County Sanitation Districts*, Los Angeles, Calif.

4:00 **Discussion**

© Harry Bullen

RUSKIN DAM AND POWER HOUSE, 55 MILES FROM VANCOUVER

BRITANNIC MINES CONCENTRATING PLANT, 15 MILES FROM VANCOUVER

Technical Sessions of the Western Professional Meeting of the Engineering Institute of Canada

Thursday—July 12, 1934, Morning and Afternoon

All events of the Engineering Institute of Canada are to be held jointly with the Society except the following technical sessions, which will be held simultaneously with those of the Society on Thursday, July 12, 1934.

9:00 **Highway Development in British Columbia**

PATRICK PHILIP, *M.E.I.C., Deputy Minister and Chief Engineer, Department of Public Works, Province of British Columbia*, Victoria, B.C., Canada.

Town Planning in the Vancouver District

W. G. SWAN, *M.E.I.C., Consulting Engineer*, Vancouver, B.C., Canada.

2:00 **The English Bay Intercepting Sewer, Vancouver**

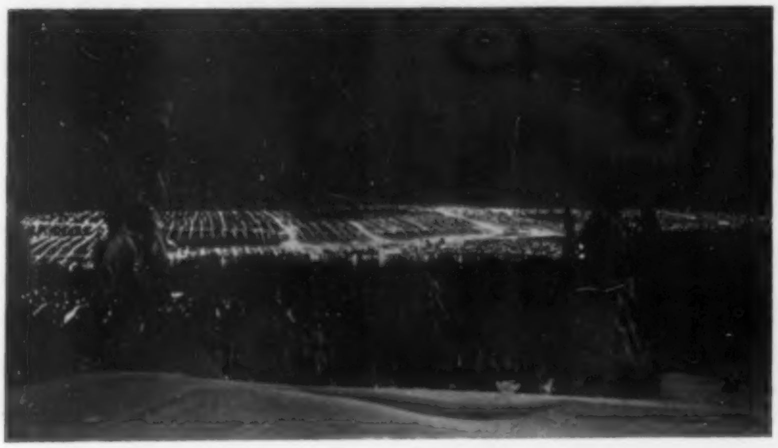
G. M. GILBERT, *A.M.E.I.C., Engineer of the Vancouver and District Sewerage Board.*

Substructure of the Reconstructed Second Narrows Bridge, Vancouver

P. L. PRATLEY, *M.E.I.C., Consulting Engineer*, Montreal, Que., Canada.

GROUSE MOUNTAIN CHALET, RENDEZVOUS FOR DINNER AND DANCE THURSDAY NIGHT

NIGHT VIEW OF VANCOUVER FROM THE CHALET



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THE SEVEN SISTERS, IN STANLEY PARK

Entertainment for the Ladies—Informal Dinner Dance

JOINTLY WITH THE ENGINEERING INSTITUTE OF CANADA

Wednesday—July 11, 1934, Afternoon

2:30 **Ladies' Drive**

Leaving the hotel at 2:30 p.m., the ladies will be taken on an automobile drive around Stanley Park, through the residential district of Vancouver, and to the University of British Columbia.

Following the drive, the ladies will be entertained at tea.

Thursday—July 12, 1934, Morning and Afternoon

No special entertainment has been provided for the ladies.

They will be free to go on shopping trips, sightseeing trips, and such, as may suit their convenience.

Members of the Ladies Committee will be in attendance at the Registration Desk to offer guidance and suggestions.

Thursday—July 12, 1934, Evening

7:30 **Informal Dinner Dance**

Automobiles will leave Hotel Vancouver at 6:00 p.m. for the Grouse Mountain Chalet, arriving about 7:30 p.m. for an informal dinner dance.

Tickets for the dinner and evening's entertainment are $2.50 each.

All-Day Boat Trip to Paper and Pulp Mill

Friday—July 13, 1934

JOINTLY WITH THE ENGINEERING INSTITUTE OF CANADA

9:00 All-Day Boat Trip for Members, Ladies, and Guests to Powell River for Inspection of a Pulp and Paper Mill

Members of the party will leave Canadian Pacific Railroad Pier "D" at 9:00 a.m. on the Canadian Pacific Railroad's S.S. *Princess Louise* for a trip by boat to the paper and newsprint mill of the Powell River Company, arriving at Powell River at 1:00 p.m. This excursion will traverse the Gulf of Georgia and the Malaspina Straits, a distance of 72 miles, to Powell River.

Following luncheon there will be an inspection of the plant, which should have many points of interest for engineering visitors, since it has a daily capacity of 650 tons.

Nearby is a 24,000-hp hydro-electric plant, which also may be inspected by interested visitors.

The party will return to Vancouver about 10:00 p.m. Dinner will be served on the boat, followed by dancing and bridge.

Tickets for the Boat Trip, including luncheon and dinner, are $5.00.

GRAND CANYON OF CAPILANO RIVER

© Western Canada Airways Ltd.

POWELL RIVER COMPANY'S PAPER AND PULP PLANT, DESTINATION OF FRIDAY'S BOAT TRIP

This Plant, Located on the Powell River About 72 Miles from Vancouver, Will Serve as an Introduction to the Pulp and Paper Industry of British Columbia. In the Left Background May Be Seen the Dam and Hydro-Electric Plant, Which Also Are of Engineering Interest and Can Be Easily Visited if Desired

Engineer Inspections—Drives—Golf

Saturday—July 14, 1934, Morning and Afternoon

JOINTLY WITH THE ENGINEERING INSTITUTE OF CANADA

9:00 Trip to British Columbia Electric Power Plant

The party will leave the Hotel Vancouver by car. Lunch will be served at the power plant at Ruskin, B.C., by courtesy of the British Columbia Electric Railway Company.

This group will then return to Vancouver at 2:30 p.m. to meet a party from the Hotel Vancouver for a visit to the Greater Vancouver Water District.

For those who prefer, golf may be arranged during the morning at any one of the city courses.

2:00 Excursion to Seymour Canyon and Falls

The remainder of the party will leave the Hotel Vancouver by car, meeting the party from Ruskin at the Second Narrows Bridge at 2:30 p.m. and driving to Seymour Canyon and Falls, where tea will be served by the Water District.

After tea, those who wish may return to Vancouver, reaching the city by 6:00 p.m., and the remainder of the party will drive to Capilano Canyon and Whytecliff, where dinner can be arranged if desired.

There is no charge for either trip.

Hotel Rates—Announcements—Reduced Railroad Rates

Railroad Rates

Excursion rates in effect at the time of the Convention will make the cost of transportation moderate. The following table gives the cost of round-trip fares and one-way Pullman rates to Vancouver from various cities:

Route	Summer Tourist Round Trip	Short Limit Summer Tourist Round Trip	Lower Berth	Upper Berth	Drawing Room	Compartment
Via Chicago, from:						
New York . . .	$138.35	$126.90*	$24.75	$19.80	$87.70	$70.00
Boston.	144.94	133.98*	25.88	20.70	92.00	73.00
Washington . .	130.45	120.75*	24.00	19.20	86.00	67.75
New Orleans . .	104.50	.....	22.50	18.00	80.00	63.50
Via Montreal, from:						
New York . . .	138.35	126.90*	25.75	20.60	90.50	72.50
Boston.	144.70	133.25*	25.75	20.60	90.50	72.50
From:						
Chicago	86.00	.....	15.75	12.60	44.50	56.00
St. Louis . . .	86.00	.....	18.00	14.60	53.50	63.00
Los Angeles . .	61.95†	57.85‡	9.50§	7.60§	34.00§	27.00§
San Francisco .	44.85†	42.55‡	6.75§	5.40§	24.00§	19.00§

* 45 days. ‡ 21 days.
† 3 months. § To Seattle.

Special Convention Tour

Arrangements have been made with the Canadian Pacific Railway for a special tour to the Convention, the group leaving Chicago at 10:00 a.m., Wednesday, July 4. Those going on this tour will spend Friday and Saturday, July 6 and 7, at Banff and Lake Louise in the Canadian Rockies, arriving in Vancouver on Monday morning, July 9.

The tour will be limited to members of the Society and their friends. Full details may be obtained by communicating with J. E. Roach, General Agent, Passenger Department, Canadian Pacific Railway, Madison Avenue and 44th Street, New York, N.Y.

Hotel Rates

Hotel	With Private Bath: Single Room	With Private Bath: Double Room	Without Private Bath: Single Room	Without Private Bath: Double Room
Hotel Vancouver . .	$4.00	$6.00 up	$3.00	$4.50
Hotel Georgia . . .	3.00	4.50–5.00	2.50	4.00
Hotel Grosvenor . .	2.00	3.00–3.50	1.50	2.50
Devonshire Apartment Hotel . . .	3.50	4.50	....	....
Ritz Hotel	2.50 up	3.50 up	....	....

These five representative hotels are situated within a radius of two blocks from the Hotel Vancouver, which is the Convention Headquarters.

As the Convention will take place during the peak of the tourist season, members are urged to make reservations early to avoid disappointment. Those intending to stay at the Hotel Vancouver will please notify the management of their connection with the Convention when making reservations.

Local Committee on Arrangements

E. A. CLEVELAND, *Chairman*
J. C. OLIVER, *Secretary-Treasurer*

C. E. BLEE
C. BRAKENRIDGE
P. H. BUCHAN
A. L. CARRUTHERS
A. E. FOREMAN
J. R. GRANT
F. LEE
L. S. MCLENNAN
H. N. MCPHERSON
J. ROBERTSON
W. O. SCOTT
H. L. SWAN
C. E. WEBB
A. S. WOOTTON
G. R. WRIGHT

Subcommittees

Registration Committee: P. H. BUCHAN, *Chairman*
Publicity Committee: E. E. CARPENTER, *Chairman*
Entertainment Committee: W. H. POWELL, *Chairman*
Program Committee: A. H. FINLAY, *Chairman*
Finance Committee: H. B. MUCKLESTON, *Chairman*
Transportation Committee: G. WALKEM, *Chairman*

Transportation

Railroad, boat, or airplane transportation is available to Alaska, Canada, or many points of interest in Washington or along the Pacific Coast. The Local Committee will be glad to assist members in planning any trip they may desire, including points of scenic or engineering interest.

Entertainment for the Ladies

Attention is directed to the program of entertainment for the ladies. The ladies are invited to participate in all the other features of the Convention which may interest them.

Invitation to Student Members

All members of Student Chapters are invited to attend and participate in all the events of the Convention.

CONVENTION HEADQUARTERS, HOTEL VANCOUVER

Ladies Committee

MRS. E. E. CARPENTER, *Chairman*

MRS. C. E. BLEE
MRS. P. H. BUCHAN
MRS. C. BRAKENRIDGE
MRS. E. A. CLEVELAND
MRS. A. H. FINLAY
MRS. A. S. GENTLES
MRS. J. R. GRANT
MRS. J. C. OLIVER
MRS. HAZEN RITCHIE
MRS. G. A. WALKEM

The program as a whole has been prepared under the direction of the Committee on Regional Meetings, HENRY D. DEWELL, *Vice-President, Am. Soc. C.E., Chairman;* and E. B. BLACK, B. A. ETCHEVERRY, RALPH J. REED and J. C. STEVENS, *Directors, Am. Soc. C.E.*

Please call on the Local Committee on Arrangements or on the Secretary's office for any service desired.

If You Drive Your Own Car to the Convention, Advise the Committee How Many Extra Passengers You Can Accommodate on Trips at Vancouver. Use Form on Page 16 ⟶

SOCIETY AFFAIRS

Official and Semi-Official

Draft of Engineering Division Code

Revised Draft of April 26, 1934, Now Receiving Consideration by NRA Boards

IT has been the effort whenever practicable to keep the membership informed of the situation with respect to the Engineering Code. None of the many drafts has been spread at length in this publication except one that seemed at the moment to have a high degree of authenticity. On April 26, too late for insertion in the May issue, there came into being the revised draft, which was submitted to the NRA and is now receiving consideration by the several boards that must pass on every Code before approval. This draft is here printed as submitted and is followed by Chairman Proctor's interpretation of its provisions.

Chapter (?). Engineering Division of the Construction Industry

ARTICLE I. DEFINITIONS

Section 1. (*a*) For the purposes of this Code the term "Engineering Division of the Construction Industry," hereinafter referred to as the "Engineering Division" or "this Division," and the term "engineering services," shall include the offer to provide, or the providing of, engineering service, in general such as, investigation, studies, surveying, planning, design, supervision, inspection, engineering testing, or laboratory analysis for the construction or anticipated construction of any private or public buildings, structures or works, or construction accessory thereto, where such service involves a knowledge of engineering laws, formulas, and practice, and a knowledge of the physical properties of the materials of construction and methods of installation, and limited to the following methods of practice, or such methods as may be later added by amendment.

(*b*) "Professional Engineering Method" shall include the providing or undertaking to provide and be responsible for any of the engineering services defined in this Section, where such services do no include constructor or contractor performance or responsibility. This method may include correlating and supervising construction and installation.

(*c*) "Engineer-Construction Method" shall include the providing or undertaking to provide and be responsible for any of the engineering services defined in this Section when combined with constructor or contractor performance or responsibility.

(*d*) The definition in paragraphs (*a*), (*b*), and (*c*) of this section shall not include engineering services necessarily adjunctive to the manufacture, sale, installation, or construction of patented devices or specialized processes or engineering design necessarily adjunctive to and predicated upon the use of particular processes or particular equipment or combinations of the same; nor shall this definition include the engineering services necessarily adjunctive to the operations of a contractor in the performance of the work called for by engineering plans and specifications.

Section 2. (*a*) For the purpose of this Chapter the term "Member of the Engineering Division," hereinafter referred to as "Member of this Division," shall include any form of organization or enterprise, or individual operating in his own behalf, providing or undertaking to provide and be responsible for, whether by formal contract or otherwise, any of the hereindefined services; provided however that

(*b*) it is recognized that the functions of an architect and of a landscape architect normally involve certain functions included in the above definition of engineering services, to an extent limited by the normal training and experience broadly characteristic of these professions, but a Member of the Architects or of the Landscape Architects Division, when exercising such functions so limited, is not intended to be considered and shall not be considered as a Member of this Division as defined in this Article. It is also recognized that the above stated functions included in engineering services involve functions included in the definition of architectural services in the Architects Division, but such functions are hereby intended to be exercised by members of the Engineering Division only to the extent limited by the normal training and experience broadly characteristic of the engineering profession. In any case where a complaint is made that a Member of this Division has provided or offers to provide services beyond the intended scope of the functions of a Member of this Division, which are the appropriate function of an Architect or Landscape Architect, but without conforming with the provisions of their respective Chapters of this Code, the complaint shall be referred to a Committee of six (6), three (3) appointed by and from the Code Authorities of each Division of the industry involved, and the decision of this Committee shall be accepted as final by Members of this Division. If either Code Authority fails to act with reasonable promptness, the matter may be referred to the Construction Appeals Board.

Section 3. A "person" as used herein is defined to mean any private or public individual, partnership, association, trust, trustee, trustee in bankruptcy, receiver, corporation, or agency.

Section 4. The term "client" as used herein is defined to be a person who engages engineering service.

Section 5. The term "contractor" as used herein is defined to mean any person contracting to perform the work called for by engineering plans and/or specifications.

ARTICLE II. ADMINISTRATION

Section 1. A Divisional Code Authority, hereinafter referred to as "the Engineering Code Authority," is hereby constituted to administer this Code within this Division.

Section 2. (*a*) The Engineering Code Authority shall consist of eleven (11) members, each of whom shall be individually representative of Members of this Division, and this Engineering Code Authority shall be appointed through the following associations by their executive committee members who are individually representative of Members of this Division. The American Institute of Consulting Engineers, the American Society of Mechanical Engineers, and the American Society of Heating and Ventilating Engineers shall each appoint one member of the Engineering Code Authority, and the American Society of Civil Engineers shall appoint five (5) members of the Engineering Code Authority representative of the Professional Engineering Method of practice in five (5) principal branches and three (3) members representative of the Engineer-Construction Method of practice. If the Administrator shall find that there are sufficient nonmembers of the aforementioned associations so that such nonmembers are entitled to representation on the Engineering Code

Authority, he may appoint one additional member to the Engineering Code Authority from, and to represent, such non-members of associations. The eleven (11) members of the Engineering Code Authority shall be subject to the approval of the Administrator.

(*b*) The terms of the appointments shall not exceed two years except that, in the event of Code continuance beyond the limit now established by law, the terms may be readjusted to insure overlapping tenures of office pursuant to a plan or method approved by the Administrator. Members are subject to replacement by the appointing agency with the approval of the Administrator.

Section 3. (*a*) The Engineering Code Authority is authorized under the supervision of and in cooperation with the Construction Code Authority in the exercise of the power conferred upon it in Section 2 (*d*) of Article IV A of Chapter I to require the registration in such manner and at such time as it may deem appropriate of all contracts for engineering service for which the charge is estimated to exceed four hundred dollars ($400.00); and, in order to defray the expenses of such registration, of the collection of the reports and the data therein required and of the administration of this code, to charge and collect, after the requirements of the Administrator have been complied with, from Members of this Division as a registration fee not to exceed one-half of one per cent of the amount of the charge for engineering services, except that the Engineering Code Authority may establish a minimum fee not to exceed five dollars ($5.00). Except as may be required for the effective enforcement of the provisions of this Code, the registration of contracts of Members of this Division shall be confidential and in their registration there shall be required only essential data, such as date, parties to the contract, location of the work, type of engineering service, and approximate charge for engineering service.

(*b*) Failure to pay any such registration fee within thirty (30) days of the receipt of due notice thereof shall constitute a Code violation provided the requirements of the administrator have been complied with and that the Administrator shall have approved an itemized budget of estimated expenses of the Engineering Code Authority and the registration fees as provided for above.

(*c*) From the funds so collected the Engineering Code Authority shall defray its expenses, such expenses as it may approve of any agencies appointed by it, and this Division's proportionate share of the expenses of the Construction Code Authority, in administering this Code, in accordance with a budget of expenses approved by the Administrator.

ARTICLE III. FAIR PRACTICES

Section 1. The provisions of Sections 2 to 12, inclusive, of this Article are adopted as rules of fair practice for all Members of this Division, and any violation of said rules shall constitute an unfair practice and a violation of this Code.

Section 2. For the purpose of administering the provisions of this Code and for the protection of the client and the Member of this Division, each Member of this Division before proceeding with an engagement for engineering services, for which his charge for engineering services is estimated to exceed four hundred ($400) dollars, shall enter into a written contract or agreement with the client for said engineering services. Such agreement shall be kept by the Members of this Division for the purposes recited in Chapter I of this Code.

Section 3. No Member of this Division shall give or offer to give any rebate, discount, bonus, fee, commission, or other valuable consideration to a client, or to a prospective client, in order to influence or procure an engagement for engineering services.

Section 4. No Member of this Division shall accept an invitation to submit a proposal for his services where such invitation has been issued by advertisement, or by any other method which indicates that price will be a principal basis for selection, except as otherwise required by law.

Section 5. Subsequent to the selection of a Member of this Division for engineering services as evidenced by a written agreement, and so long as such an agreement is in effect, no other member shall offer to provide any competitive engineering services.

Section 6. (*a*) No Member of this Division shall issue or permit to be issued on his behalf plans and/or specifications without the signature of the engineer under whose direction the plans and/or specifications are prepared, conspicuously subscribed thereto, as evidence of the engineer's technical responsibility for their preparation and use.

(*b*) In all engineering services as defined in Article I, where individuals are placed in positions supervisory of the engineering, and/or in positions from which the principles of the engineering may be dictated, such individuals shall be engineers. Delegation of authority by engineers so placed shall be only to engineers or engineering assistants.

(*c*) The term "engineer," as used in connection with this Section shall mean any individual, whether or not he be a Member of this Division; who is engaged in the functions described in Section 1 of Article I; and who is registered or licensed in accordance with the provisions of law to practice engineering, or any branch thereof, within any state or territory of the United States; or who is admitted to or is eligible by reason of technical training for admission to membership in established national professional engineering associations in a grade which requires, as a condition of membership, active practice in engineering for not less than eight (8) years, with evidence of competency to be placed in responsible charge of engineering, active practice, and responsible charge of engineering to be construed as fulfilled only when embodying the design of engineering works and/or the direction of engineering functions.

(*d*) The term "engineering assistant," as used in connection with this Section, shall mean any individual, not qualified as an "engineer" as hereindefined, who is employed to render technical or other services of a professional nature under the direction of an engineer.

Section 7. If the character of the work requires architectural, or specialized engineering, or other technical services, supplementary to those normally rendered by the Member of this Division, it is his duty to employ or recommend for employment by or on behalf of the client such specialists and to coordinate their work.

Section 8. No Member of this Division shall submit his designs or drawings and/or specifications for contractors' bids until he has developed adequate preliminary information and essential pertinent data. Such designs or drawings and/or specifications shall be complete and sufficient so that it will be unnecessary for any contractor or other person to complete or to provide further engineering services except shop or working drawings and engineering services necessarily adjunctive to the operations of a contractor in performing the work called for by engineering plans and/or specifications. A Member of this Division shall observe fairness in making known all the requirements for the work for which he is responsible to all bidders alike and shall furnish adequate information to all bidders.

Section 9. (*a*) No Member of this Division shall acquire, own, or otherwise benefit by the purchase, sale, or ownership in any real estate, or business interest which may be affected by an engineering work for which he shares responsibility, except with the full knowledge and consent of his client.

(*b*) No Member of this Division shall accept compensation or other valuable consideration from any source connected with the work, other than from his client, except with the full knowledge and consent of his client.

Section 10. No Member of this Division shall specify, direct, invite, or accept engineering designs, studies, sketches, or inspection of construction materials from a contractor, subcontractor, or supplier of materials, interested in the work or in the bidding on any element of the work, where such services are the responsibility and duty of the Member of this Division. This does not prohibit the acceptance of the customary shop and working draw-

ings for the work, or conferences with contractors, subcontractors, or suppliers of materials concerning designs or details of construction, or the use of technical trade information issued by producers, nor does this prohibit any member operating under the Engineer-Construction Method from full cooperation with his construction or operating organizations and/or with any member practicing under the Professional Engineering Method.

Section 11. (*a*) No Member of this Division shall make a charge to the client for engineering services at less than the cost of such services as determined in paragraph (*b*) of this Section. This provision is not for the purpose of indicating the fair or proper charge, but is for the purpose of setting a minimum limit below which charges shall not be permitted for engineering services.

(*b*) Cost shall be considered as the actual cash expenditures chargeable to the engineering services in question, plus charges for the services of the principals for the time they actually spend on the engineering services at an adequate rate and not less than the highest salary paid to any member of the organization or employee actually engaged on the engineering services in question, plus cost of specialists employed by the Member of this Division, plus overhead applied to the payroll cost and charges for principals' services; the overhead factor to be determined by a method prescribed by the Engineering Code Authority and approved by the Administrator, and to include all items of office expense such as the clerical force, insurance, and rent, and a proper share of engineering and other payroll costs and of expenses not assignable to definite engagements, and to include a proper share of the cost of the principals' time not assignable to definite engagements to compensate for the readiness to serve and the professional responsibility assumed. Prior to such prescription by the Engineering Code Authority a reasonable overhead factor shall be determined by each Member of this Division from his own cost records.

(*c*) Since it is often imperative that a client determine in advance of operations definite limits of cost, including cost of engineering services, in order to conform to an appropriation or to permit financial arrangements, a member, without violating paragraph (*a*) of this Section, may agree in advance to a percentage or lump sum, when such charges by the member are in accordance with charges which are authoritatively established for such services or when such charges represent an honest effort to meet the conditions of paragraph (*b*) of this Section.

(*d*) No Member of this Division shall represent that he will provide, nor shall he provide, engineering services free or at less than cost as defined in paragraph (*b*) of this Section, except that he may provide without charge preliminary advisory services prior to an engagement provided the recipient of these services has expressed the intent to engage the member's further engineering services if and when such further services are required for the anticipated construction with which said preliminary services are concerned. In the event that such further services are engaged, the charge shall include a proper charge for the preliminary advisory services.

Section 12. No Member of this Division shall misrepresent the character of his engineering services or those of a competitor, make an unfair or untrue statement as to a competitor, or take action tending to induce cancellation of a competitor's contract or agreement.

ARTICLE IV. REFERENCE TO PROVISIONS OF CHAPTER I

The provisions of Sections 7 (*a*) and 10 (*b*) of the Act, which are set forth in Sections 1 and 6, respectively, of Article VIII of Chapter I of this Code, are specifically incorporated herein by reference with the same force and effect as if set forth herein in full; all other provisions of Chapter I of this Code, except as herein provided, apply within this Division with the same force and effect as if set forth herein in full.

ARTICLE V. REVIEW OF ACTS OF THE ENGINEERING CODE AUTHORITY

If the Administrator shall determine that any action of the Engineering Code Authority or any agency thereof may be unfair or unjust or contrary to the public interest, the Administrator may require that such action be suspended to afford an opportunity for investigation of the merits of such action and further consideration by the Engineering Code Authority or agency pending final action, which shall not be effective unless the Administrator approves or unless he shall fail to disapprove after thirty (30) days' notice to him of intention to proceed with such action in its original or modified form.

ARTICLE VI. MODIFICATIONS

Subject to the provisions of Section 2 (*c*), of Article IV, B, of Chapter I of this Code, the provisions of this Chapter, except as to provisions required by the Act, may be modified on the basis of experience or changes in circumstances, such modifications to be based upon application to the Administrator and such notice and hearing as he shall specify, and to become effective on his approval.

ARTICLE VII. EFFECTIVE DATE

This Chapter of this Code shall become effective on the 30th day after the approval of this Chapter pursuant to the Act.

Influence of the Proposed Engineering Code

An Interpretation of How It Will Operate and Whom It Will Affect

Back of every official document there is a mass of discussions, case analyses, mutual understandings, and rulings, out of which becomes evident the intent of the final written words as they are to be applied in administration. Mr. Carlton S. Proctor, Chairman of the Code Group, here presents his interpretation of the scope and application of the Code for the Engineering Division of the Construction Industry. This interpretation has no legal status but does indicate clearly the conditions which the group drafting the Code has understood to be fundamental.

The Code for the Engineering Division of the Construction Industry has been in course of preparation for nearly ten months and is now being submitted to the Government officials for approval. It may help to clarify thought on the subject to recall at this time why the Federal authorities considered an Engineering Code necessary; how it came about that the American Society of Civil Engineers undertook to prepare it; what are the general provisions proposed; and who are affected by it.

It will be remembered that the National Industrial Recovery Act, which became a law in June 1933, was an emergency measure designed to stimulate reemployment of labor at a living wage and to foster fair competition among employers. Industry was told that it could govern itself under Federal supervision, and to this end Codes of Fair Practice were to be established to effect this regulation. A Code of the Construction Industry at once came up for consideration, and the Construction League of the United States undertook this task. It was decided that the most satisfactory way would be for the Construction League to prepare a basic code of general provisions which would apply to all the divisions of the industry, and then to have each division write a code which would apply only to itself. Members of a division would then be governed jointly by the Code of General Provisions, to be known as Chapter 1, and by their specific Divisional Chapter.

Chapter 1 was approved by President Roosevelt on January 31, 1934, and became effective on March 2, following. In it the Construction Industry is defined to include the designing as well as construction of buildings and fixed structures. This reference to design involves the practice of architecture and of engineering, and for this reason it has been ruled by the Government that for code purposes those practicing those professions shall be grouped as Divisions of the Construction Industry, with certain limitations. Those affected therefore are those who are engaged competitively in the business of proffering or rendering architectural or engineer-

ing service to a client in the form of design for new or anticipated new construction. This at once eliminates the Federal Government, states, cities, and all other political subdivisions; the railroads, the utilities, contractors, manufacturers, institutions, and all others until they shall engage in competition on the basis of design for projected construction.

All codes are written to ensure fair practice among employers and so are strictly employers' codes. The Government has endeavored to have them prepared by organizations national in extent and truly representative of the employers in the group affected. There was no organization in the engineering profession that exactly fulfilled these requirements and so the NRA officials recognized the American Society of Civil Engineers as being the most nearly representative to undertake the task of writing the engineering chapter. The chapter to be written was to be solely to ensure fair practice among employers, and the legal department of the Government ruled that the Code could not specify wages or hours of work other than for common labor. However, it does contain a definition of "engineer" and "engineering assistant," indicating them as having a preferred relationship to that of common labor. The Code further provides that engineering specifications and drawings must prominently display the signature of the engineer who is responsible for them and also stipulates that when principles of engineering are to be dictated, this shall be done by an engineer, or if delegated by him to others, it must be to other engineers or engineering assistants. The general subject of wages and hours of work for other than common labor has been taken care of in Chapter 1, "General Provisions," which provides for collective bargaining.

With the understanding that it would be undertaking nothing prejudicial to the interests of employee members of its organization, the Society accepted the obligation, and a Code Committee to represent the main branches of the profession was formed.

It was recognized by the Code Committee that the document it was to prepare must have a precise form, as it is to become, in effect, a law. Every effort was made to get constructive ideas on the subject from the entire engineering profession, and the draft as finally submitted is the result of many months of discussion, during which certain interpretations of the enabling act and certain interpretations of the intent of that act were made by NRA officials and accepted as fundamentals.

Chapter 1, which is the chapter of general regulations, contains the provision, common to all codes, which prohibits child labor, arranges for collective bargaining, and provides for maximum hours of work and a minimum wage for common labor. No one under the age of 16 may be employed, and office employees are limited to 40 hr of work per week over a period of four weeks. This does not apply to employees engaged in professional, executive, and supervisory work, as they are exempted. The minimum wage refers to unskilled labor only and is fixed at 40 cents per hr. The collective bargaining clause provides for collective bargaining through regional agreements. A single region may be considered as the entire United States, but as there are wide differences in the cost of living as between the less populated areas and the larger cities, it is probable that the regions will be many and relatively small. In some parts of the country a region may include several states, but in more thickly populated areas it is probable that it will be confined to a state or even to a city and its metropolitan district. The regional agreements also will apply only to employees of the employers who come under the Code.

The Chapter which pertains exclusively to the members of the Engineering Division defines those to whom the Code is applicable, arranges for administration, and stipulates rules of fair practice. The members of the division are all those who are engaged competitively in the business of proffering or rendering engineering service to a client in the form of design for new or anticipated new construction, and is designed as primarily for those generally spoken of as "in private practice" or for those organizations rendering engineering services later combined with construction performance. However, those who are normally operating under another code come under the provisions of this Code and so become Members of the Engineering Division when they compete with engineers doing work as defined above. Thus a contractor or a manufacturer who is operating under his own code in the normal prosecution of his work, automatically comes under the Engineering Code the moment he does engineering work which fulfills the above conditions. A manufacturer may employ all the engineers he wants to design the articles he is to manufacture and be unaffected by this Code, but if he extends his engineering work so that he is engaged competitively in the business of engineering service, as defined above, he becomes a Member of the Engineering Division Code and must abide by it in the conduct of the competitive work that he is doing. The Code cannot say that he may not enter into competition with engineers, but it can say that in doing so he must make an adequate charge to the client for the cost of such work.

On the other hand, engineers may render services in the form of expert testimony, in valuations, or appraisals; in personal consultations or in teaching, and as long as their work does not contain design for construction, the Code does not touch them.

The Code is to be administered by a Code Authority consisting of 11 members, each of whom shall be individually representative of members of the Division.

Provision is made for the registering of all contracts for engineering service which come within the scope of the Code and for which a charge is estimated to exceed four hundred dollars ($400.00). The expense of administering the Code Authority is to be met from registration fees not to exceed one-half of one per cent of the amount of the charge for engineering services.

Adequate provisions have been made to ensure fair practice among the employers who are Members of the Engineering Division, but the provisions of course apply to them alone. Each code must be self-contained. It is contrary to the Act to put anything into a code that will tell those under another code, or, in fact, that will tell any others not under the Code in question, what they shall or shall not do. For those border line cases such as work done by Architects, Landscape Architects, and Engineers, reciprocal clauses have been worked out which are identical in wording and which are incorporated in all three codes. These clauses ensure the right of each profession to function as it does now, with the understanding that each one will employ the other for work peculiarly its own. In this way the interests of each professional group are safeguarded without trying to set definite limits in the twilight zone where the work of the professions merge.

This reciprocal arrangement was arrived at after many conferences and the most careful consideration and is typical of the painstaking study that has been given by all concerned to the many perplexing problems that have presented themselves.

The very limiting nature of the Act itself has frequently made it impossible to write into the Code many of the things that have been ardently advocated by some enthusiasts. The Code which is now before the NRA for approval is a document which represents the earnest efforts of a Code group to safeguard the interests of the Engineering profession and at the same time to raise it to an ever higher plane.

CARLTON S. PROCTOR
Chairman, Code Committee

May 16, 1934

Secretary's Abstracts of Intermediate Meeting of the Board of Direction, on May 11, 1934

THE BOARD of Direction met at 6:30 p.m., at the Engineers Club, New York, N.Y. Present were President Harrison P. Eddy, in the chair; George T. Seabury, Secretary; and Messrs. Barbour, Gregory, Horner, Lupfer, McDonald, Perry, Sanborn, Sherman, Trout, and Treasurer Hovey.

There was informal discussion regarding the number of applications that are being received for admission to the Society, the number of members who are paying their dues, and the general financial condition of the Society.

There was also discussion relative to the proposed Engineering Division Code, with an analysis of the scope of its application.

The Board adjourned at 7:50 p.m. to meet at Vancouver, B.C., Canada, on Monday, July 9, 1934, at 10 a.m.

Society's Name Used Without Authorization

INFORMATION has recently reached Headquarters that a firm of photographers in Montreal has been using the Society's name in soliciting business. Its method appears to be to take portraits

without cost and then make a charge for prints when copies are desired. Use of a society's name to indicate apparent approval of a concerted procedure is not uncommon, but such approval has not been given by the Society. Any person claiming the contrary should be assumed to be soliciting business under false pretenses.

New Committee on Flood Protection Data

AT ITS MEETING in January 1934, the Board of Direction authorized the formation of a new Society committee, that on "Flood Protection Data." It consists of the following members, who have accepted the President's appointment to serve: Gerard H. Matthes, Chairman; Frederick H. Fowler, Robert E. Horton, Ivan E. Houk, Charles W. Kutz, Charles W. Sherman, and Daniel C. Walser, all Members of the Society.

Committee to Promote Hydraulic Research

UNDER authorization from the Board of Direction at its January meeting, President Eddv has appointed a new Society Committee on Hydraulic Research. The personnel of this group, which is now to begin its labors, is taken entirely from the membership of the Society and is under the chairmanship of J. C. Stevens, including in addition C. E. Bardsley, and E. W. Lane, Members Am. Soc. C.E., and L. G. Straub, H. D. Vogel, and Chilton A. Wright, Associate Members Am. Soc. C.E.

All these men are qualified by training and experience to help in this important work. Each is familiar with the theoretical and practical aspects of hydraulic research, a subject in which civil engineers are taking great interest.

Appointments of Society Representatives

BERTHOLD F. HASTINGS, M. Am. Soc. C.E., and CHARLES A. HOWLAND and PHILIP H. CARLIN, Associate Members Am. Soc. C.E., were Society delegates to the meeting of the American Academy of Political and Social Science, held in Philadelphia on April 13.

HARRISON P. EDDY, President Am. Soc. C.E., and ALONZO J. HAMMOND, Past-President Am. Soc. C.E., are Society representatives to the Construction League of the United States; and FRANK A. RANDALL and WILSON T. BALLARD, Members Am. Soc. C.E., have been appointed to serve as alternates to the league.

C. E. GRUNSKY, Past-President Am. Soc. C.E., and FRANKLIN THOMAS, M. Am. Soc. C.E., have accepted appointments to serve as Society delegates at the summer meeting of the American Association for the Advancement of Science, to be held in Berkeley, Calif., June 18–23, 1934.

F. S. MERRILL and H. A. THOMAS, Members Am. Soc. C.E., will represent the Society at the winter meeting of the American Association for the Advancement of Science, to be held in Pittsburgh, Pa., from December 27, 1934, to January 2, 1935.

News of Local Sections

ALABAMA SECTION

On April 13 the Alabama Section held a meeting at the Redmont Hotel in Birmingham, Ala. The feature of the occasion was a paper presented by N. W. Dougherty, Professor of Civil Engineering at the University of Tennessee. Professor Dougherty's topic was "Professional Consciousness," which included a discussion of the advantages and disadvantages of the Engineers' License Law. Then Hugh Martin, of Miller and Martin, architects of Birmingham, gave a talk on the subject of the Alabama Architects' License Law.

ARIZONA SECTION

The spring meeting of the Arizona Section, which was held at Tucson on April 14, involved an all-day program of entertainment. Various business matters were discussed, and the reports of several committees were heard. On the technical program the speakers were Ralph J. Reed, a Director of the Society and consulting engineer of Los Angeles, and Richard Stephens, Jr., an engineer with the Metropolitan Water District of Southern California. In the evening a dinner was enjoyed in the company of members of the University of Arizona Student Chapter.

CENTRAL ILLINOIS SECTION

The Central Illinois Section held its regular bimonthly meeting on April 20 in Champaign, Ill., with 31 members and guests present. An excellent résumé of current hydraulic projects was given by Prof. J. J. Doland, of the Department of Civil Engineering, of the University of Illinois.

CENTRAL OHIO SECTION

There were 27 present at a luncheon meeting of the Central Ohio Section held in Columbus on May 17. The guest speaker, Prof. K. W. Stinson, gave an interesting talk on the experimental and practical application of streamlining to the design of railroad trains and motor cars.

CINCINNATI SECTION

A dinner, attended by 42 members and guests, preceded the meeting of the Cincinnati Section held on April 11. There were 60 in attendance at the meeting. After the presentation of several committee reports, the speaker of the evening, W. R. Woolrich, was introduced. Mr. Woolrich, who is a mechanical engineer with the Tennessee Valley Authority, gave an interesting account of the most important aspects of the industrial, agricultural, sociological, and economic development of this project. The annual election of officers, which took place at this time, resulted as follows: John S. Raffety, President; Clifford M. Stegner, Vice-President; and William W. Carlton, Secretary-Treasurer.

CONNECTICUT SECTION

There were about 40 present at the annual meeting of the Connecticut Section held at the University Club in Hartford on April 24. During the business session the following officers were elected for the ensuing year: John C. Tracy, President; Henry R. Buck, Vice-President; and Joseph P. Wadhams, Secretary-Treasurer. An illustrated address on the construction of Boulder Dam was given by Robert Ridgway, Past-President of the Society.

DAYTON SECTION

The regular meeting of the Dayton Section for March took the form of a luncheon meeting held at the Engineers Club on the 19th, with 20 present. After the reading of a report of the Prize Contest Committee, M. L. McKercher, of the American Bitumuls Company, was introduced. Mr. McKercher gave an illustrated talk on the subject, "The Use of Asphalt Emulsion in Highway Construction and Maintenance."

Members of the Dayton Section, who attended the meeting held on April 16, had the privilege of hearing Robert Hoffmann, a Director of the Society and consulting engineer on public works for the City of Cleveland, discuss recent activities of the Society. Then Charles H. Paul gave a brief but interesting talk on his work as a member of the Mississippi River Commission. The attendance numbered 30, including 7 members from the University of Dayton Student Chapter.

FLORIDA SECTION

In conjunction with a session of the Florida Engineering Society, the Florida Section of the Society held its annual meeting in Jacksonville, on April 12, 13, and 14. The business session of the

Section took place at a dinner meeting on April 12, at which recent correspondence with the Society was read and the action taken on various matters was discussed. The technical part of the program was given on April 13 and 14. Numerous papers on subjects of engineering interest were presented on this program, and several inspection trips were enjoyed by the members. A general smoker for the men pleasantly occupied one evening, and the ladies present had an opportunity to enjoy golf, bridge, and other diversions.

Georgia Section

Numerous interesting meetings were held by the Georgia Section during the late winter and early spring. On April 2 F. H. McDonald, a Director of the Society, spoke on several topics of interest, including the Annual Meeting of the Society, held in New York, N.Y., in January, and the work of the U. S. Coast and Geodetic Survey in Georgia. A joint luncheon meeting of the Georgia Section of the Society and of local branches of the American Society of Mechanical Engineers and the American Institute of Electrical Engineers was held at the Atlanta Athletic Club on April 16. A five-reel official Government motion picture of construction work on Boulder Dam was shown on this occasion. There were 79 present.

Metropolitan Section

At the annual meeting of the Metropolitan Section, which was held in New York, N.Y., on May 16, an interesting lecture on "Meteorology and Its Importance to the Engineer" was delivered by Dr. Armin K. Lobeck, Professor of Geology at Columbia University. He spoke of the origin and distribution of rainfall and prevailing winds, the effect of temperature, the action of storms, and the theories of climatic changes. This lecture was extensively illustrated.

At its conclusion the final program of the Junior Branch of the Metropolitan Section, comprising its annual Effective Speaking Contest, was held. As a result, awards were made to the following Juniors: first prize of $15 to J. James Knox; second prize of $10 to C. W. E. Schroeder; and honorable mention to Samuel C. Clark and D. J. Liccione.

Following the reading of the annual report by the secretary of the Section, the report of the Nominating Committee was received and, in accordance with it, the following officers were elected for 1934–1935: Ole Singstad, President; E. R. Needles, Vice-President; and W. J. Shea, Secretary. After adjournment at 10:45 refreshments were served. The attendance was about 225.

North Carolina Section

The result of the annual election of officers for the North Carolina Section has resulted as follows: S. H. Wright, President; D. S. Abell and J. L. Becton, Vice-Presidents; and Harold C. Bird, Secretary-Treasurer.

Northwestern Section

At a meeting of the Northwestern Section held on April 12, Lorenz G. Straub, of the College of Engineering of the University of Minnesota, addressed the group on "Modern Experimental Design of Elastic and Hydraulic Structures." His talk was illustrated by slides and motion pictures of various engineering projects, in which experiments on models played an important part in the design and analysis of the structures.

Philadelphia Section

The more important aspects of the U. S. Engineers' report on the Delaware River were presented at a meeting of the Philadelphia Section held on March 21. This session attracted the largest attendance of any of this year's technical meetings, as there were 64 present at the dinner and 142 at the meeting. Discussion of the report was opened by Col. Earl I. Brown, of the Corps of Engineers, U. S. Army, who made the report.

The annual Student Chapter meeting of the Philadelphia Section was held on April 18, with a total attendance of 118 at the meeting and 84 at the dinner that preceded it. Representatives were present from the Student Chapters of the University of Pennsylvania, Pennsylvania Military College, Drexel Institute, the University of Delaware, and Bucknell University. Members of these Chapters provided entertainment and enlivened the event by surprise stunts. An interesting address on the subject, "Some Oddities in Mechanics," was given by C. J. Tilden, Strathcona Professor of Engineering Mechanics, at Yale University.

Portland (Ore.) Section

There were 58 present at a meeting of the Portland, Ore., Section held on April 19. After several business matters were attended to, the speaker of the evening, J. C. Stevens, was introduced. Mr. Stevens is a member of the engineering firm of Stevens and Koon, of Portland, and a consulting engineer on the Bonneville project now under construction on the Columbia River. The subject of his address was "Backwater Effect of the Bonneville Dam," which was illustrated by diagrams that had been prepared for the occasion. Work on dams on the Ohio River was then discussed briefly by Claude I. Grimm, Chief Engineer of the Bonneville Dam Section for the U. S. War Department. A general discussion from the floor followed.

San Diego Section

On April 26 the San Diego Section held its regular dinner meeting at the Churchill Hotel. A talk on the subject, "The Highway System of California," was given by E. E. Wallace, District Engineer of the California State Division of Highways. In this talk, which was illustrated by three reels of motion pictures, Mr. Wallace outlined the history of the present highway system and the developments planned for the future.

San Francisco Section

There were 118 present at the regular meeting of the San Francisco Section held at the Engineers Club on February 20. Besides a business session there was a program of entertainment, which included an interesting exhibition of magician's tricks. The technical program was devoted to a survey of the Civilian Conservation Corps and its work. The speakers on this subject were Capt. Arthur J. McChrystal, of the Ninth Corps Area, and E. W. Kramer, Regional Engineer of the U. S. Forest Service. At the meeting held on April 17, the attendance numbered 170. Different aspects of the general topic, "Earthquakes and Engineering Structures," were presented by the three speakers. These were Erle L. Cope, consulting engineer and member of the Advisory Board to the State Division of Architecture; L. S. Jacobsen, Professor of Mechanical Engineering at Leland Stanford University; and John D. Galloway, consulting engineer of San Francisco.

Tacoma Section

There were 31 present at the monthly dinner meeting of the Tacoma Section, held on March 12. The principal speaker was J. D. Ross, superintendent of the Light Department of Seattle, Wash., who spoke on the topic, "Power Development in the Pacific Northwest." Exceptionally fine colored slides were shown in connection with Mr. Ross's talk.

A meeting of the Tacoma Section held on April 9 was devoted to discussion of the subject of planning. The principal address on the program was given by E. F. Banker, Director of Conservation and Development of the State of Washington, who described the recent work of the state in the field of surveying and the development of power on the Columbia River and its effect on agricultural production in the state. Other speakers were Roy J. Sharp, president of the Committee of One Hundred; P. W. Bourgaize, president of the City Planning Commission; and G. Frank Rhodes, secretary of this commission.

Virginia Section

The annual meeting of the Virginia Section was held at Virginia Beach, Va., on March 17. This was a joint meeting with sessions of the Engineers Club of Hampton Roads and local branches of the American Society of Mechanical Engineers and the American Institute of Electrical Engineers. During the business session the Section elected officers for the coming year as follows: W. D. Faucette, President; and J. L. Newcomb, R. Keith Compton, and W. D. Tyler, Vice-Presidents. P. A. Rice continues as Secretary-Treasurer. After the business meeting luncheon was served, and an interesting address, by Louis Dreller, Commander, U. S. Navy, was heard. The afternoon was devoted to golf and to swimming in the indoor salt water pool of the Cavalier Hotel, and in the evening approximately 150 gathered for dinner. The speaker at this function was L. H. Mitchell, of the U. S. Bureau of Reclamation, who gave an interesting and informative lecture on Boulder Dam. This talk was illustrated by motion pictures.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for July

When the President announced that the Government, through the Civilian Conservation Corps (CCC), was to undertake an extensive program of reforestation, it was immediately contended by some that this work would not keep 300,000 men busy. However, the President and his advisers had in mind many projects besides tree planting. An article in the April issue described the development of Hayes State Park, Michigan, by the CCC. In the July number, E. W. Kramer, M. Am. Soc. C.E., will discuss the engineering activities of the CCC in California, where during the past winter there were 150 camps, housing 30,000 men. In addition to engineering construction activities, these men were engaged on such work as making trails and fire breaks, controlling forest pests, and building telephone lines.

In another article planned for the July issue, E. L. Chandler, M. Am. Soc. C.E., briefly points out the principal features of a new type of hydraulic turbine installed in the new unit of the Government-owned hydro-electric plant of Sault Ste. Marie, Mich. The building of this unit is the first step in the reconstruction of the existing plant. The feature of the propeller-type turbine is the method used to increase efficiency by adjusting the tilt or pitch of the blades with variations in both head and load. The tests on the unit surpassed the guarantees, and the results appear to justify the care used in both design and construction.

Water works engineers will find interest in the article by Clarence J. Velz, Assoc. M. Am. Soc. C.E., on "The Influence of Temperature on Coagulation," an interpretation of data obtained from experiments in water treatment conducted in New Jersey. He found that when either alum or chlorinated copperas was used as a coagulant, best floc formation and color removal were obtained for a given dosage when the temperature of the raw water was reduced between 2 and 14 C. While this result seems to be at variance with the usual conception that chemical activity increases with rise in temperature, Mr. Velz's analysis of his data is convincing.

As explained by A. S. Mittag, Assoc. M. Am. Soc. C.E., the best-equipped aviation field south of Texas is rapidly approaching completion in the Canal Zone near the Balboa entrance to the Panama Canal. Construction of Albrook Field was begun by pumping mud from the Canal prism into an alligator-infested swamp. Hangars and machine shops for the planes, concrete aprons for warming up and taking off, and three landing areas constitute the facilities of the field. In addition, there are well-planned quarters for the personnel.

Another article to appear in the July issue is by Thomas J. Noland, Jr., Jun. Am. Soc. C.E., who describes the equipment used in measuring the heat of hydration of cements for Boulder Dam. In preparing the specifications for the cement in Boulder Dam, an extensive study was made of scores of commercial and laboratory cements. One phase of the investigation was the actual determination of the amount of heat of hydration. The major results of this phase of the investigation are given by Mr. Noland.

In addition, the July issue will contain the article by Alfred Brahdy, M. Am. Soc. C.E., on some of the construction phases of the New York Independent Subway System. This article was announced for the June issue but was crowded out by the Convention program and by the draft of the Engineers' Code. Other articles are on hand for July, to be used as space permits.

National Conference on Laboratory Uses of Models

Throughout the teaching and engineering professions much interest has been evidenced in the forthcoming Models Conference to be held in Ithaca, N.Y., from June 19 to 21, in connection with the Annual Meeting of the Society for the Promotion of Engineering Education. The program has been arranged to include papers on both structural and hydraulic models to be presented by speakers competent to give the very latest developments in the art of designing and analyzing engineering structures. The aim of the conference is to bring together from all over the country a group of men doing pioneer work in the field of materials, hydraulics, sanitation, and structures, for a pooling of experience, for mutual guidance, and for stimulation of interest in models.

Sessions begin on the morning of June 19, with registration, a welcome to the campus of Cornell University, and an address on the laws of similitude. In the afternoon the structural section will discuss the deformeter method of model analysis, photo-elasticity, and the various types of loaded models. At the same time, those interested in hydraulics will attend illustrated lectures on the work being done at four laboratories in the United States. These are the U. S. Waterways Experiment Station, at Vicksburg, Miss., the Iowa Institute of Hydraulic Research, the University of Maine Laboratory, and the Alden Hydraulic Laboratory, at Worcester, Mass.

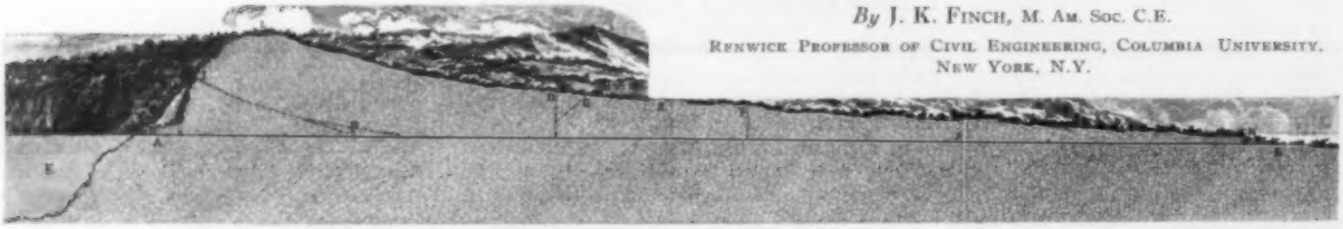
On Wednesday, June 20, there will be presented papers on specific applications of models to structural designs, such as building frames, suspension bridges, continuous-span concrete-arch bridges, and dams. Other papers will discuss the use of photo-elastic methods, of the deformeter machine, and of models in engineering instruction. The hydraulics section will also consider specific problems in the solution of which models have been helpful. These include the movement of bed sediment, the control of rivers, erosion below dams, the design of settling basins, the study of filter sands, and the design of side channel spillways.

The conference closes at noon on Thursday, June 21, with a review of the present tendencies in engineering education. This subject will be considered from the standpoint of both the structural and the hydraulic section. The speakers include: C. M. Allen, M. Am. Soc. C.E., of Worcester Polytechnic Institute; George E. Beggs, M. Am. Soc. C.E., of Princeton University; T. H. Evans, Jun. Am. Soc. C.E., of Yale University; Gordon M. Fair, M. Am. Soc. C.E., of Harvard University; M. M. Frocht, of Carnegie Institute of Technology; H. J. Gilkey, M. Am. Soc. C.E., of Iowa State College, Fred L. Plummer, Assoc. M. Am. Soc. C.E., of the Case School of Applied Science; J. C. Rathbun, M. Am. Soc. C.E., of the College of the City of New York; K. C. Reynolds, M. Am. Soc. C.E., of Massachusetts Institute of Technology; J. L. Savage, M. Am. Soc. C.E., U. S. Reclamation Service, Denver; E. W. Schroder, M. Am. Soc. C.E., of Cornell University; Hale Sutherland, M. Am. Soc. C.E., of Lehigh University; W. J. Sweetser, of the University of Maine; J. T. Thompson, M. Am. Soc. C.E., of the Johns Hopkins University; and W. E. Wickenden, President of the Case School of Applied Science. Throughout the conference there will be on display an extensive group of exhibits and demonstrations of model studies.

Accommodations for those who come alone as well as for those accompanied by their families will be provided at Cornell University in certain of the women's dormitories. The rates to be charged for such rooms will be very moderate, and meals will be available at the Union Building, similarly at a moderate cost, so that the necessary expenses for those attending the conference will be very small. No registration fee of any kind will be required. The conference is open to all who are interested. All arrangements at Cornell should be made through the Arrangements Committee, of which E. N. Burrows, M. Am. Soc. C.E., Professor of Bridge Engineering at Cornell University, is chairman. Additional information concerning the conference may be obtained by addressing Fred L. Plummer, Assoc. M. Am. Soc. C.E., Professor of Structural Engineering at the Case School of Applied Science, Cleveland Ohio, who is chairman of the conference.

Lowering the Level of the Alban Lake

By J. K. Finch, M. Am. Soc. C.E.
Renwick Professor of Civil Engineering, Columbia University, New York, N.Y.



Tunnel and Construction Shafts for Draining the Alban Lake
Lake Level Was Just Below "I" When Drainage Began, in 396 B.C.

According to tradition Rome was founded in 753 B.C. Some two and a half centuries later, in 509 B.C. to be exact, the successful struggle of the citizens of Rome against the Tarquin kings led to the establishment of the Republic. This form of government endured for almost five centuries, that is, until the time of Caesar, about the beginning of the Christian era, when the Empire came into being.

The great engineering accomplishments of early Rome included the construction of the Servian Wall for the protection of the city; the beginning of the drainage construction which later developed into the Cloaca Maxima, or storm sewer of Rome; the rediscovery of the arch, which became the most important structural form used in Roman construction; and the discovery of pozzuolana, or volcanic ash cement.

The early years of the Republic were filled with the Samnite Wars and the struggle with Carthage, so that construction of muncipal works was somewhat delayed. In this period, however, three important engineering undertakings are to be noted: the start of the Great South Road, or Appian Way, the building of the first aqueduct, and the lowering of the Alban Lake. Of these, the latter was the earliest by some forty years, which makes it the first great engineering work of the Republic and one of the first great works of Roman times.

About 16 miles south of Rome, and forming part of the circle of hills that surround the city, is the Alban Mountain. One or two of the less important water supplies for the ancient city originated in this area. However, practically all the greater supply systems, after following the hillside from Tivoli southward, crossed from this Alban Mountain region to the city, where high ground was found. This reduced the required height of the arch construction used for carrying the aqueduct across the relatively flat country between the mountains and the Seven Hills of Rome.

The Alban Mountain, like much of the area near Rome, is of volcanic origin, and in the depressions of its huge crater lie two beautiful and picturesque lakes. This region, which is supposed to be the site of Alba Longa, the mother city of Rome, later became a favorite resort of wealthy Romans. In draining Lake Nemi one of the two lakes in the crater, the Italian government recently uncovered two ancient pleasure barges of this period, when the Alban Mountain was a rich man's paradise.

Both these lakes were supplied at an early date with tunnel outlets, by means of which their water level was stabilized. Little is known of the outlet of Lake Nemi but that of the Alban Lake, according to tradition and early records, dates back to the year 396 B.C. Both the ancient Roman authors, Livy and Cicero, repeat the legend connected with this work.

Apparently the Alban Lake, completely landlocked and with no natural outlet other than the crater lip, rose to an unprecedented level in 398 B.C. At this time Rome was carrying on a war with the Etruscans and was besieging one of their last strongholds, Veii, just north of Rome.

Fig. III

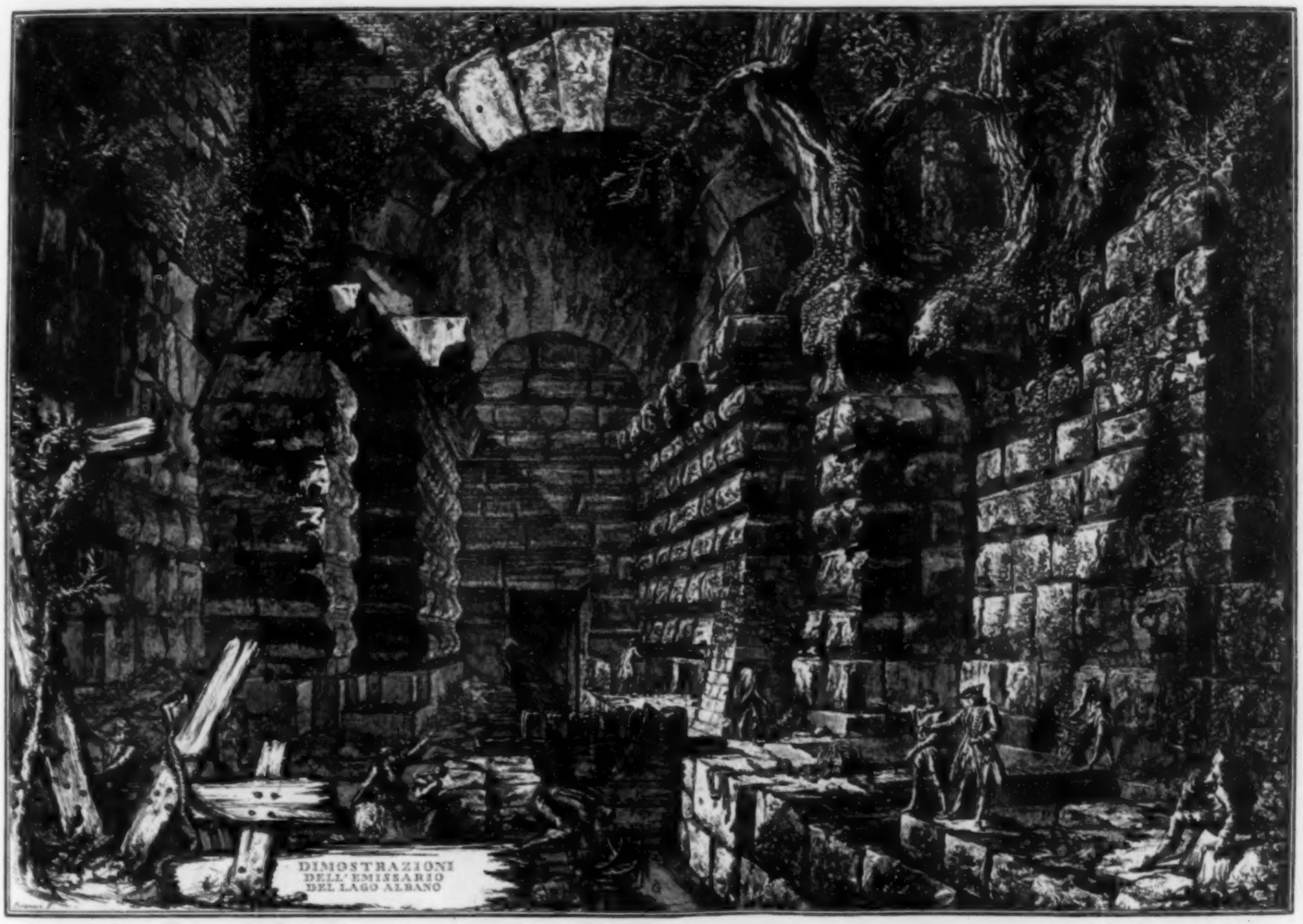
Control Structure at the Intake Portal
Built After the Lake Level Was Lowered

According to the legend, an Etruscan seer had declared that Veii would not fall until the Alban Lake was drained. When this was reported in Rome, it is said that a young Roman warrior kidnapped the seer and brought him before the Senate, where he repeated his prophecy.

Even this, however, failed to convince the Roman leaders who, to assure themselves of the value of the information, decided to send a deputation to consult the Greek Oracle of Apollo at Delphi. It is related that the prediction was confirmed and, furthermore, that the lake was drained by a tunnel built in one year. It is also said that a similar work brought about the downfall of Veii, for the victorious Romans entered the stronghold by means of a tunnel driven under the citadel. Finally the debt to the oracle was paid by sending one-tenth of the booty collected at Veii as an offering to Delphi.

When an attempt is made to discover the actual facts and to justify the drainage of the Alban Lake on economic grounds, certain features of the work make the reconstruction of this historical episode somewhat difficult. There seems to have been little economic reason for the construction of a tunnel purely for drainage purposes. The Alban Lake has steep shores, and variations in its level would not flood valuable agricultural lands. Cicero, however, offers a hint which may indicate the true reason for the construction. He says, in effect, that the water was drawn from the lake to irrigate valuable lands on the outer slope of the mountain—"The work was intended to benefit the suburban farms and not to secure the safety of Rome." Again, it is certain that an elaborate diversion structure was erected at the outlet. Probably this was done at a later time, but the structure was intended for use in connection with irrigation.

The ancient legend about the work, however, does involve interesting elements. It is possible that the deputation sent to Delphi not only consulted the oracle but also brought back Greek engineers to direct the tunnel operations. Roman engineering works are so extensive and imposing that the Roman engineer has received almost the entire credit for establishing the profession of master builder in the Mediterranean area. As a matter of fact, many of Rome's triumphs in construction were founded on technical knowledge imported from Greece. The Greeks had established tunnel surveying

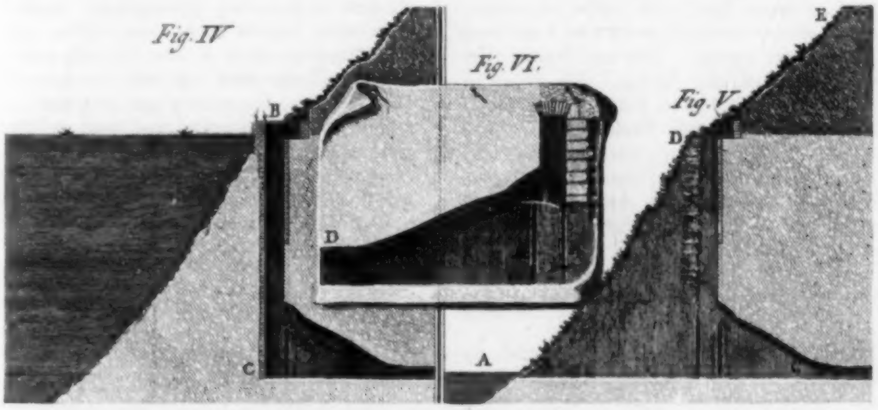


Intake Portal Structure

on a sound basis at least as early as 600 B.C., and ancient Greek engineers had achieved a wide reputation as tunnel experts. It would be quite natural for the Roman delegation to Delphi to secure such expert services for a type of undertaking which was new to Roman workers.

Whatever the answer to this problem may be, it is certain that the Alban Lake tunnel is strikingly similar to early Greek works as far as the methods employed in its excavation are concerned. It passes through a solid hill of the volcanic *peperino* stone, broken here and there by basaltic masses, at a level of 430 ft below the crater lip. In height it varies from 5 to 10 ft, and nowhere is it less than $3^1/_2$ ft wide. The walls show chisel marks about one inch wide, which indicate that the work was accomplished by the slow process of chiseling, or picking out, the rock.

As was almost invariably the practice in ancient times, the tunnel was driven not only from the end headings, but also from additional headings at the bottom of intermediate shafts. In fact, in some of these ancient works the length of such shafts and inclined adits exceeds that of the tunnel itself. Vitruvius, for example, recommends shafts at intervals of about 300 ft! This practice was obviously followed to expedite the work of tunnel excavation, as labor was plentiful.



Intake Shaft Sunk at the Lake's Edge
Progressive Destruction of Shaft from the Top Allowed Water Level to Be Gradually Lowered

At the Alban Lake outlet the remains of three vertical shafts and one inclined shaft are discernible. The latter was probably used to furnish easier access for the workers and for the removal of excavated material. These sloping shafts, later known as *cuniculi*, or "rabbit burrows," which provided access to the central part of a tunnel, were shorter than a vertical shaft when the mountain to be pierced was steep and high.

Driven at the elevation to which it was desired to lower the lake, the tunnel at the beginning of the drainage operation was many feet lower than the original lake level. From Piranesi's engravings it is apparent that a shaft was sunk to the grade of the tunnel at the high-water level of the lake and lined with masonry. By the process of excavation, the top of the shaft was first lowered to permit the lake to flow into it, and thence out through the tunnel. This process continued as the lake level was lowered until the shaft disappeared and the level of the lake was that of the tunnel. Thereafter the lake level was maintained at tunnel elevation.

TUNNEL OUTLET STRUCTURE AND DISTRIBUTING WORKS

As in the case of so many ancient works, authorities differ in regard to the dimensions of this undertaking. The total length of the channel is about $1^1/_2$ miles, but the length of the tunnel proper is much less. One report gives it as 1,300 yd.

Remains of interesting outlet and regulating works are still visible. These are of uncertain date, but probably belong to a much later period than the tunnel itself. At the lake end, there is a channel with cross walls, gratings, a screen for rubbish, and a settling basin, similar to those used on the Roman aqueducts for the removal of sand and mud. At the entrance to the tunnel there is an elaborate portal structure designed to protect the channel from falling rocks. Where the tunnel emerges, on the other side of the hill, there is a long, covered outlet reservoir, with five outlets, presumably for distributing water to the irrigation channels.

While no modern detailed study and drawings of this interesting work have been made, the famous Italian etcher and engraver of the eighteenth century, Giambattista Piranesi, has left us a number of beautiful engravings of the Alban Lake work as it existed in his day. One of Piranesi's engravings [reproduced on the Page of Special Interest in this issue] shows the tunnel outlet structure and distributing works for irrigating the land on the outer slopes of the mountain. The small sketches introduced at the top of this engraving show the details of dividing the water inside the outlet structure, and a section through the Alban Lake and its drainage tunnel. This and many other engravings of the Alban Lake drainage works are included in Piranesi's "Descrizione e Desegno dell' Emissario del Lago Alban" in the volume, *Antichita d'Albano e di Castel Gandolfo*. [The accompanying illustrations, as well as that on the Page of Special Interest, are reproduced from a copy of this work owned by the Avery Library of Columbia University, New York, N.Y.]

Piranesi executed over a thousand of these interesting and artistic engravings of Roman ruins, many of them of large size. It is only fair to warn the reader, however, that Piranesi did his best to give a grand scale to his beloved Roman antiquities, as he wanted them to appear very great and imposing. Accordingly, in his prints he made the figures, which are so characteristic of his work and serve the same purpose as the foot rule in a modern engineering photograph, smaller than they should have been. The men he pictures are only $4^1/_2$ or 5 ft tall, which enhances the size of the structures. Nevertheless, his prints are of great interest to the historian and also to the print collector, for they show the remains of an ancient and wonderful engineering and architectural age, and they do it with a feeling and artistic skill admired by print lovers the world over.

Daniel Guggenheim Medal for Aeronautic Achievement

FOR SUCCESSFUL pioneering and achievement in aircraft manufacturing and air transport, William Edward Boeing has been awarded the Daniel Guggenheim Medal for 1934. This award was made by a board consisting of 8 members from the United States and 7 from other countries. All 15 members are men of high standing in aeronautic science and engineering. The 7 foreign countries represented on the board are Canada, England, France, Germany, Holland, Italy, and Japan.

The medal was established in 1928 by the Daniel Guggenheim Fund for the Promotion of Aeronautics. It is sponsored jointly by the American Society of Mechanical Engineers and the Society of

Automotive Engineers, each of which appoints four members of the Board of Award. Medals have been awarded previously to Orville Wright, of the United States; Ludwig Pradtl, of Germany; Frederick William Lanchester, of England; Juan de la Cierva, of Spain; and Jerome Clarke Hunsaker, of the United States. The medal is awarded not oftener than annually.

An early pioneering effort of Mr. Boeing was the construction of the flying boat which carried the first air mail flown on this continent by a private contractor. This was on the line between Seattle and Victoria, B.C. When the Government was faced with the problem of getting airplanes for the Air Corps during the World War, the Boeing plant was equipped in experience and personnel to get production under way promptly. During and after the World War, the Boeing plant expanded until it is now one of the largest airplane plants in the world. Training planes, pursuit planes, large armored attack planes, bombers, torpedo planes, and long-distance patrol planes have been produced by it for the Government. These types vary in weight from 2,000 to 27,500 lb. A large and important share of all the airplanes used by the U. S. Government for both naval and military purposes has been built at the Boeing factory.

Under Mr. Boeing's direction, the transport companies became the first to fly passengers at night on regular schedule over long distances, the first to operate tri-motored passenger transports over long distances and at night, and the first to be fully equipped with two-way radio telephones. He deserves particular credit for the development of an outstanding aviation manufacturing and transport organization throughout the United States. It was his vision and willingness to spend his own money which resulted in the formation of one of the best manufacturing and transport organizations in the world.

Reading for Self-Improvement of Young Engineers

A number of worth-while suggestions came out of the studies of the Committee on Professional Training under the Engineers' Council for Professional Development, as noted in the annual report of that organization dated October 1933. Of special significance is that part of the committee's report covering a "Tentative Reading List for Junior Engineers," here given in full.

Following is a list of books recommended by a number of eminent men, distinguished in the engineering profession. Systematic reading of worth-while books adds breadth and vision to the background of an engineer and should be considered as part of the intellectual development designed to fit young engineers for full professional recognition.

The list is prepared on the basis of a generally accepted classification. There are about one hundred titles. Over a period of about five years, twenty-five of these books might be selected and read, with the limiting recommendation that the selection be made which will include at least one book in each classification. Selections should be made in accordance with the individual engineer's most vital interest.

Natural Science

Humanism and Science, by C. J. Kaiser, Columbia University Press, 1931
What Industry Owes to Chemical Science, by Pilcher and Jones, D. Van Nostrand Co., 1931
Grammar of Science, by Karl Pearson (London), A. & C. Black, 2nd edition, 1900
Human Life as the Biologist Sees It, by Vernon Kellogg, Henry Holt & Co., 1922
Mysterious Universe, The, by Sir James Jeans, Macmillan Co., 1932
New Conceptions of Matter, The, by C. G. Darwin, Macmillan Co., 1931
Outline of Science, by John A. Thomson, G. P. Putnam's Sons, 1922
Philosophical Basis of Biology, by John Scott Haldane, Doubleday, Doran & Co., 1931
Universe in the Light of Modern Physics, by Max Planck. W. W. Norton & Co., 1931
Van Loon's Geography, by H. W. Van Loon, Simon and Schuster, 1932
Metallurgy and Its Influence on Modern Progress, by Sir Robert Hadfield, D. Van Nostrand Co., 1931

Philosophy, Including Religion

Pleasures of Life, The, by John Lubbock
Coming Religion, The, by Nathaniel Schmidt, Macmillan Co., 1930
Introduction to Reflective Thinking, An, by Columbia Associates in Philosophy, Houghton Mifflin Co., 1932
Man's Rough Road, by A. G. Keller. Yale University Press, 1932
Story of Philosophy, by William Durant, Simon and Schuster, 1926
Straight and Crooked Thinking, by R. H. Thouless, Simon and Schuster, 1932
What Men Live By, by R. C. Cabot, Houghton Mifflin Co., 1914
Guide Through World Chaos, by G. D. H. Cole, Alfred Knopf, 1932
Meaning of a Liberal Education, The, by Everett D. Martin, W. W. Norton & Co., 1926
Education and the Good Life, by Bertrand Russell
Plato's Republic, *Companion Classics*, Charles Scribner's Sons, 1928

Economics and Sociology

America Comes of Age: *A French Analysis*, by André Siegfried, Harcourt, Brace & Co., 1927
Constructive Citizenship, by L. P. Jacks, Doubleday, Doran, 1928
Can Business Prevent Unemployment? by S. A. Lewisohn, *et al.*, Alfred Knopf, 1925
Causes of Industrial Unrest, by J. A. Fitch, Harper & Bros., 1924
Economic Stabilization in an Unbalanced World, by A. H. Hansen, Harcourt, Brace & Co., 1932
Farewell to Reform, by John Chamberlain, Liveright, 1932
Industrial Economics, by D. S. Kimball, McGraw-Hill Book Co., 1929
Industrial Society, 3 vols., by Leon C. Marshall, University of Chicago Press, 1929–1930
Introduction to Sociology, An, by Dawson and Gettys, Ronald Press, 1929
Labor Problems in the U. S., by E. E. Cummins, D. Van Nostrand Co., 1932
Mexico: *A Study of the Two Americas*, by Stuart Chase, Macmillan Co., 1931
Middletown: *A Study in Contemporary American Culture*, by R. S. and H. M. Lynd, Harcourt, Brace & Co., 1929
Our Changing Civilization: *How Science and the Machine Age Are Reconstructing Modern Life*, by John H. Randall, Frederick Stokes Co., 1929
Recent Social Trends in the U. S., *Report of President Hoover's Research Committee on Social Trends*, 2 vols., McGraw-Hill Book Co., 1932
Recovery: *The Second Effort*, by Sir Arthur Salter, Century Co., 1932
Road to Revival, by F. C. James, Harper and Bros., 1932
Taming Our Machines, by Ralph Flanders, Richard R. Smith, Inc., 1931
Planned Society, A, by George Soule, Macmillan Co., 1932
Human Traits and Their Social Significance, by Irwin Edman, Houghton Mifflin Co., 1920
Man as Psychology Sees Him, by Edward E. Robinson, Macmillan Co., 1932
Practical Psychology, by Edward S. Robinson, Macmillan Co., 1926
Misuse of Mind, by Karin Stephen, Harcourt, Brace & Co., 1922
Psychology for Executives, by Elliott D. Smith, Harper & Bros., 1928
Readings in Industrial Psychology, by Moore and Hartman, D. Appleton & Co., 1931
Psychology at Work, by P. S. Achilles, McGraw-Hill Book Co., 1932

Business and Industrial Management

Young Man in Business, The, by H. L. Davis, John Wiley & Sons, 1932
Art of Business Reasoning, The; Art of Business Thinking, The, by Schnackel and Sprecker, John Wiley & Sons, 1930
Banking, by B. Foster, Alexander Hamilton Institute, 1923
Business Leadership, by H. C. Metcalf, Isaac Pitman & Sons, 1930
Economic Control of Engineering and Manufacturing, by F. L. Eidmann, McGraw-Hill Book Co., 1931
Economics of Overhead Costs, Studies in the, by J. M. Clark, University of Chicago Press, 1923
Factory Management, by Bangs and Hart, Alexander Hamilton Institute, 1930
Organization Engineering, by Henry Dennison, McGraw-Hill Book Co., 1931
Principles of Industrial Organization, by D. S. Kimball, McGraw-Hill Book Co., 1913

Literature

Enjoyment of Literature, The, by Jay B. Hubbell, Macmillan Co., 1929
Essays in Persuasion, by J. M. Keynes, Harcourt, Brace & Co., 1932
Emerson's Journals, The Heart of, Ed. by Bliss Perry, Houghton Mifflin Co., 1926
Morte d'Arthur, 2 vols., by Sir Thomas Malory, E. P. Dutton & Co., 1932
Oxford Book of English Verse (1910), Ed. by Sir Arthur Quiller-Couch
English Literature, by W. N. C. Carlton, American Library Association Reading Course

History

Modern Democracies, 2 vols., by Viscount Bryce, Macmillan Co., 1924
History of the American People, by Willis M. West, Allyn and Bacon, 1926
Historical Evolution of Modern Nationalism, by C. J. H. Hayes, Richard R. Smith, 1931
Rise of American Civilization, 2 vols., by C. A. and M. Beard, Macmillan Co., 1927
Outline of History (new and revised), by H. G. Wells, Garden City Publishing Co., 1931
History of Ten Years (1918–1928), by R. L. Buell, Macmillan Co., 1929
Soviet Russia, by W. H. Chamberlain, Little, Brown & Co., 1930
Northward Course of Empire, by Vilhjalmur Stefansson
Epic of America, The, by James T. Adams, Little, Brown & Co., 1931

Biography

Travels of Marco Polo, Ed. by G. B. Parks
Americanization of Edward Bok, by Edward Bok, Charles Scribner's Sons, 1920
John A. Brashear, Autobiography of, Ed. by W. L. Scaife, American Society of Mechanical Engineers, 1924
Leonardo da Vinci, Life of, by Clifford Bax, Macmillan Co., 1932
Education of Henry Adams, by Henry Adams, Houghton Mifflin Co.

LIVES OF THE ENGINEERS, by Samuel Smiles (London), John Murray
LIFE AND LETTERS OF JOHN HAY, 2 vols., by W. R. Thayer, Houghton Mifflin Co., 1915
IMMIGRANT TO INVENTOR, by Michael Pupin, Charles Scribner's Sons, 1923
ABRAHAM LINCOLN, LIFE OF, by Lord Charnwood, Henry Holt & Co., 1916
LINCOLN STEFFENS, AUTOBIOGRAPHY OF, Harcourt, Brace & Co., 1931
GEORGE WESTINGHOUSE, LIFE OF, by Henry G. Prout, Charles Scribner's Sons, 1922
TWO YEARS BEFORE THE MAST, by R. H. Dana, Houghton Mifflin Co.

FINE ARTS

MODERN PAINTERS, by Ruskin, E. P. Dutton Co. and Longmans, Green and Co.
HOW TO APPRECIATE PRINTS, by Frank Weitenkampf, Charles Scribner's Sons, 1932
ENJOYMENT OF ARCHITECTURE, by T. F. Hamlin, Charles Scribner's Sons, 1921
THEATRE, Ed. by E. J. R. Isaacs, Little, Brown & Co., 1927
LISTENING TO MUSIC, by Douglas Moore, W. W. Norton & Co., 1932
SCOPE OF MUSIC, THE, by Percy Buck, Oxford University Press, 1924

FICTION

WAR AND PEACE, by Count Tolstoi
GIANTS IN THE EARTH, by O. E. Rolvaag
GOOD EARTH, THE, by Pearl Buck
ETHAN FROME, by Edith Wharton, Charles Scribner's Sons
HENRY ESMOND, by William Thackeray, Grosset & Dunlap
VANITY FAIR, by William Thackeray
KIDNAPPED, by R. L. Stevenson
KIM, by Rudyard Kipling, Doubleday, Doran & Co.
LORD JIM, by Joseph Conrad, Doubleday, Doran & Co.
WAY OF ALL FLESH, by Samuel Butler, Grosset & Dunlap
WUTHERING HEIGHTS, by Emily Brontë, Grosset & Dunlap

GENERAL WORKS

FRONTIERS OF KNOWLEDGE, by Jesse Lee Bennett, American Library Association Reading Course
PUBLIC OPINION, by Walter Lippman, Macmillan Co., 1927
REPRESENTATIVE GOVERNMENT, by H. J. Ford, Henry Holt & Co., 1924
PHILOSOPHY OF STYLE, by Herbert Spencer, D. Appleton & Co., 1911
ENGLISH FOR EVERYBODY, by J. M. Miller
SENTENCES AND THEIR ELEMENTS: *Theory and Practice of Technical Writing*, by Samuel Earle, *et al.*, Macmillan Co., 1912
HOW TO LIVE ON 24 HOURS A DAY, by Arnold Bennett
THE EFFICIENT LIFE, by Luther H. Gulick
HUMAN BODY AND ITS CARE, THE, by Morris Fishbein

International Congress of Applied Mechanics

THE FOURTH International Congress of Applied Mechanics will be held in Cambridge, England, from July 3 to 9, 1934. Twenty-one of the papers to be presented at this session are by American engineers. These are being grouped under four general headings, as follows: (1) rational mechanics, including vibrations of structures and machines; (2) mechanics of fluids, including turbulence, the boundary layer, heat transfer, and compressible fluids; (3) materials, including elasticity, plasticity, fatigue, and crystal structure; and (4) water waves, including resistance and stability of ships and seaplanes. The American participating committee for this congress is the Applied Mechanics Division of the American Society of Mechanical Engineers, 33 West 39th Street, New York, N.Y.

NEWS OF ENGINEERS

From Correspondence and Society Files

RAUL J. F. LUCCHETTI has accepted an engineering position in the Department of Public Welfare of the New York City Works Division, with offices in New York City.

RICHARD DE CHARMS is now senior inspector for the Bureau of Yards and Docks of the U. S. Navy, with headquarters in Quantico, Va.

GEORGE B. RICHARDSON, engineer and contractor of Decatur, Ill., has assumed the presidency of the Richardson Construction Company, Inc., of the same city.

LAWRENCE F. WRIGHT, formerly an engineer and estimator for Miller-Daybill and Company, of New York, N.Y., has accepted the position of superintendent of construction for the Groves-Quinn Corporation, a firm of general contractors of the same city.

LEO F. CARDEN has accepted a connection with the New Mexico State Highway Department in the capacity of project engineer.

RALPH P. THOMPSON is now employed as a computer by the U. S. Coast and Geodetic Survey, with headquarters in New York, N.Y.

FRANK L. REYNOLDS has resigned as engineer in charge of field operations for Henry C. Irons and Sons, of New York, N.Y., to become construction engineer for Barr, Irons and Lane, Inc., of the same city.

GEORGE S. WARD has taken the position of foreman of C.C.C. Camp 1408—p 61, at Georgetown, S.C.

HERBERT R. PIRKNER, who was formerly concrete designer for the Western Electric Company, of New York, N.Y., is now cultural foreman of C.C.C. Camp No. 4, at Orange, N.J.

ORRIN H. PILKEY has accepted employment with the Poirier and McLane Corporation, of New York, N.Y., in the capacity of construction engineer.

CURTIS H. GUERNSEY has resigned as county-city engineer of Cherokee, Okla., to become an associate engineer in the U. S. Bureau of Biological Survey on water problems connected with migratory bird refuges. His headquarters are in Washington, D.C.

ROY W. MORSE has taken a position as assistant engineer with the consulting engineering firm of W. C. Morse Company, of Seattle, Wash.

WALTER F. SHATTUCK, JR., formerly assistant superintendent of Starrett Brothers, Inc., of Glencoe, Ill., has now become associated with Shattuck and Layer, architects, of Chicago.

C. CLARKE KEELY has severed his engineering connection with the Stone and Webster Engineering Corporation, of Los Angeles, Calif., to accept a position as junior engineer with the Metropolitan Water District of Southern California, with headquarters in the same city.

H. CLINTON RAPP is now superintendent of quarries for the Mathieson Alkali Works, of Saltville, Va.

WILLARD F. BEAN, formerly with the New England Public Service Company, of Errol, N.H., has taken a position as transitman in the State Highway Department of New Hampshire, with headquarters in Lancaster, N.H.

RICHARD H. JAMISON has resigned as assistant hydraulic engineer for the California State Division of Water Rights to become engineer in charge of the Water Survey of Ventura County, California, with offices in Santa Paula, Calif.

HAROLD G. BROWNE has accepted employment with the Metropolitan Water District of Southern California in the capacity of junior engineer. His headquarters are in Los Angeles, Calif.

HAROLD O. SJOBERG has been made a partner in the firm of N. H. Sjoberg and Son, general contractors, of San Francisco, Calif.

FREMONT W. SLATTERY has taken a position as project engineer in the U. S. Indian Service, with headquarters in Farmington, N.Mex.

WILLIAM C. CRAM, JR., is now assistant to the Assistant Secretary of the Treasury, with offices in Washington, D.C. He was formerly district manager for the Allied Engineers, Inc., of Atlanta, Ga.

ASHLEY B. TAYLOR has accepted an engineering connection with the Kansas City Power and Light Company, in Kansas City, Mo.

THOMAS L. HODGES is now employed by the U. S. Coast and Geodetic Survey on triangulation work in several Southern states. His headquarters are in Asheville, N.C.

GEORGE P. BISCHOF has severed his connection with the Brooklyn Eastern District Terminal, of Brooklyn, N.Y., to become a sales engineer for the F. MacGovern Corporation, of New York, N.Y.

ERNEST D. HENDRICKS has taken the position of Senior Claims Engineer in the New York State Department of Public Works. His office is in Albany, N.Y.

LOUIS H. LOCKWOOD is now an engineering inspector for the Reliance Advertising Company, of New York, N.Y.

NORMAN B. NEWCOMB has joined the staff of the Port of New York Authority in the capacity of junior engineer.

Thomas R. Jacobi, formerly with the U. S. Bureau of Reclamation, is now employed by the Ulen Management Company, of New York, N.Y., as maintenance engineer on the public utilities of the City of Sao Luiz, Maranhao, Brazil.

William Nelson Carey has accepted the position of State Engineer of Minnesota for the Public Works Administration, with offices in St. Paul, Minn.

W. Bion Moore has severed his connection with the U. S. Coast and Geodetic Survey to become a research engineer for the Federal Power Commission. His headquarters are Washington, D.C.

Clayton B. Neill has severed his connection with the Loveland Engineers, Inc., to become engineer and assistant manager of the Coronado Water Company, and chief engineer of the Sweetwater Water Corporation, of National City, Calif.

Robert A. Thompson has accepted an appointment as State Engineer of Texas under the Federal Emergency Administration of Public Works, with headquarters in Fort Worth, Tex.

P. J. Freeman, formerly consulting engineer for the Pittsburgh Testing Laboratory, has now taken the position of Principal Materials Engineer in the Engineering Service Division of the Tennessee Valley Authority. His offices are in Knoxville, Tenn.

M. E. Phillips has taken a position as engineer field aide with the U. S. Biological Survey on land surveys for fish and game refuges. Formerly he was in charge of surveys and design for the Quartermaster, U. S. Army, at Fort Riley, Kans.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From April 10 to May 9, 1934, Inclusive

Additions to Membership

Alexander, Melville Richmond (M. '33), Asst. Engr., Div. of Office Engr., The Panama Canal, Box 452, Balboa Heights, Canal Zone.

Applegate, Daniel Webster (Jun. '33), Insp., U. S. Bureau of Reclamation, Nyssa, Ore.

Baird, Walter Douglas (Jun. '33), 1722 North Henderson St., Dallas, Tex.

Bennett, Robert Noble (Jun. '33), 512 Third St., Findlay, Ohio.

Biermann, Arthur Edward (Jun. '33), 8646 Trafford Lane, St. Louis, Mo.

Buford, Jack William (Jun. '33), 3 Hammond St., Cambridge, Mass.

Carr, Giles Lacey (Jun. '33), Chandlerville, Ill.

Carter, Elden Wuest (Jun. '33), Eng. Draftsman, U. S. Bureau of Reclamation, Owyhee Project, Nyssa (Res., 1735 North East 47th Ave., Portland), Ore.

Chang, Han Ying (Assoc. M. '34), Chf. Secy. and Member, Yellow River Comm., Kaifeng, Honan, China.

Dack, Bruce M. (Jun. '33), 1717 Winona Boulevard, Los Angeles, Calif.

de Jong, John (Jun. '33), Chainman-Rodman, State Highway Dept., Troutdale, Ore.

Eardley, James John (Jun. '33), 1228 Thieriot Ave., New York, N.Y.

Eden, Edwin Winfield, Jr., (Jun. '34), Chf. of Party, Middlesex County Mosquito Extermination Comm., Metuchen (Res., 244 Benner St., Highland Park), N.J.

Enander, Carl Alfred (Jun. '33), 244 First St., Dunellen, N.J.

Flanigan, Pierce John, Jr., (Jun. '34), 4114 Groveland Ave., Baltimore, Md.

Fluck, Miles Dietrich (Jun. '33), Telford, Pa.

Friedman, Joseph Lyon (Jun. '34), 3400 Wayne Ave., New York, N.Y.

Hall, George Yu Young (Jun. '33), 18 John St., San Francisco, Calif.

Harding, Robert Carnegie (Jun. '33), Route 3, Box 240, Tucson, Ariz.

Hatch, Loranus Pendleton (Jun. '34), 118-C Holden Green, Cambridge, Mass.

Hawes, Lorin Lindley (Jun. '34), Asst. Hydr. Engr., Braden Copper Co., Coya, Rancagua, Chile.

Heinz, Otto, Jr., (Jun. '33), 2418 South 71st St., Philadelphia, Pa.

Hiegl, George, Jr., (Jun. '33), 216 East Eleanor St., Philadelphia, Pa.

Hsu, Shih-ta (M. '33), Executive Member and Chf. Engr., North China River Comm.; Member, Technical Committee, Hai Ho Impvt. Comm. (Res., 11 Via Roma, Italian Concession), Tientsin, China.

Janssen, Allen Sheeley (Jun. '33), Instr., Civ. Eng. Dept., Univ. of Idaho, Moscow, Idaho.

Johnson, Kenneth Gordon (Jun. '33), 265 East 50th St., Seattle, Wash.

Jones, Edwin Alfred (Jun. '33), 4551 Seventeenth Ave., N.E., Seattle, Wash.

Joyce, Joseph Peter, Jr., (Jun. '33), 5006 Washington Boulevard, Chicago, Ill.

Kelly, Will Lee (Assoc. M. '34), With County Engrs. Office, Court House, Fort Worth, Tex.

Kitch, Richard Benson (Jun. '33), 213 North Harvey Ave., Oak Park, Ill.

Kopp, Alvin Henry (Jun. '33), Instr. and Supervisor of Traffic Survey, Newark Coll. of Eng., 365 High St. (Res., 22 Seymour Ave.), Newark, N.J.

Kramer, Robert William (Jun. '34), 849 Fairwood Ave., Columbus, Ohio.

Lee, Harry Harrison, Jr., (Jun. '34), Borough Engr., 333 North Pennsylvania Ave., Mórrisville, Pa.

McGraw, William Harold (Jun. '34), Junior Bridge Constr. Engr., San Francisco-Oakland Bay Bridge, San Francisco (Res., 492 Chetwood St., Oakland), Calif.

Mainardi, Pompey (Jun. '33), 5 Virginia Ave., Paterson, N.J.

Malone, James Robert (Jun. '33), 1312 Gregson St., Durham, N.C.

Maltman, Alexander (Assoc. M. '33), Acting Asst. Engr., Dept. of City Transit (Res., 6983 Weatham St., Germantown), Philadelphia, Pa.

Mason, Alvin Hughlett (Jun. '34), Chf., Astronomical Party, U. S. Coast and Geodetic Survey (Res., The Maury, 19th and G Sts., Apartment 32), Washington, D.C.

Mayer, Sam Soma (Jun. '33), 1335 Madison Ave., New York, N.Y.

TOTAL MEMBERSHIP AS OF MAY 9, 1934	
Members	5,749
Associate Members	6,274
Corporate Members	12,023
Honorary Members	18
Juniors	3,159
Affiliates	107
Fellows	4
Total	15,311

Mochamer, Thomas Richard (Jun. '33), 117 East Centre St., Centralia, Pa.

Murphy, Edward Joseph (Jun. '33), 375 East 205th St., New York, N.Y.

O'Brien, Francis Joseph (Jun. '34), Supervisor, Materials Div., Tennessee Val. Authority; 2022 West Cumberland Ave., Knoxville, Tenn.

Omsted, Harald (Assoc. M. '34), 105 South Catalina Ave., Pasadena, Calif.

Penna, Leo Amaral (Jun. '34), Asst. Hydr. Engr., Emprezas Electricas Brasileiras, S.A., Caixa Postal 883, Rio de Janeiro, Brazil.

Penner, Frank Joseph, Jr., (Jun. '33), 673 Lexington Ave., Brooklyn, N.Y.

Pennotti, Angelo Raymond (Jun. '33), 734 East 165th St., New York, N.Y.

Pugh, Wallace Raymond (Jun. '33), 1210 Pleasant St., Boulder, Colo.

Rees, Harold (Jun. '34), 843 South Durbin St., Casper, Wyo.

Sample, Mathias William (M. '34), Mine Supt., Chile Exploration Co., Chuquicamata, Chile.

Suenderman, Henry (Jun. '33), Colma, Calif.

Taul, Horace Wayne (Jun. '33), 3933 Jensen Ave., Calwa, Calif.

Wong, Chro-chun (Jun. '33), Care, Edward G. Yee, 516 Cheever Court, Ann Arbor, Mich.

Ziegler, Robert Newton (Jun. '33), Rodman, Hamilton County San. Eng. Dept.; 7601 Montgomery Rd., Cincinnati, Ohio.

Membership Transfers

Baker, Paul Wallace (Jun. '29; Assoc. M. '33), Asst. Constr. Engr., Met. Utilities Dist., Utilities Bldg. (Res., 202 South 37th St.), Omaha, Nebr.

Barnett, Joseph (Assoc. M. '26; M. '34), Senior Highway Design Engr., Bureau of Public Roads, 1725 F St., N.W., Washington, D.C.

Darling, James Arthur (Jun. '27; Assoc. M., '34), Draftsman, The Port of New York Authority, 111 Eighth Ave., New York, N.Y. (Res., 173 Vassar Ave., Newark, N.J.).

Gill, Grayson Woodward (Assoc. M. '27; M. '34), Archt., 3625 Shenandoah St., Dallas, Tex.

Hanrahan, Francis John (Jun. '30; Assoc. M. '34), Instr., Mechanics and Materials of Constr., Pennsylvania State Coll., 103 Engineering "A," Pennsylvania State Coll., State College, Pa.

Havelin, Julius Edward (Jun. '23; Assoc. M. '33), Engr., Mech. Eng. Div., Philadelphia Elec. Co. (Res., 2123 Spruce St.), Philadelphia, Pa.

Hooper, Olcott Lorin (Jun. '24; Assoc. M. '34), Asst. Hydr. Engr., E. M. Gilbert Eng. Corporation, 412 Washington St., Room 1002, Reading, Pa.

HUTCHINSON, ROBERT PADDOCK (Jun. '27; Assoc. M. '34), Structural Designer, Olympic Forest Products Co., Port Angeles (Res., 505 Boylston Ave., North, Seattle), Wash.

LORENCE, WALTER ERNEST (Assoc. M. '30; M. '34), Capt., Corps of Engrs., 1st New York Engr. Dist., 39 Whitehall Bldg., New York, N.Y.

MURCHISON, EDWARD TOWLER (Assoc. M. '22; M. '33), Chf., Water and Sewer Section, Dept. of Operation and Maintenance, A Century of Progress, Administration Bldg., Burnham Park, Chicago, Ill.

RICE, PAUL PRESTON (Jun. '29; Assoc. M. '34), State Representative, U. S. Coast and Geodetic Survey; Instr., Civ. Eng., Lafayette Coll., 103 Pardee Hall, Lafayette Coll., Easton, Pa.

SMITH, FRANK MILLER, JR., (Jun. '27; Assoc. M. '34), Associate Prof., Civ. Eng., North Texas Agri. Coll., Arlington, Tex.

VERNON, ABNER JOSEPH (Jun. '27; Assoc. M. '34), 102 Garrison Ave., Jersey City, N.J.

RESIGNATIONS

APPELDER, TAYLOR PIGGATT, Jun., resigned April 20, 1934.

AROYAN, HAGOP HAROOTUNE, M., resigned April 9, 1934.

BURGARD, JOHN WILLIAMS, Jun., resigned April 26, 1934.

HELLER, JOHN CHARLES, Assoc. M., resigned April 10, 1934.

HEPBURN, GEORGE HUNT, M., resigned April 24, 1934.

KIDD, WALTER HERBERT, Jun., resigned April 13, 1934.

LAUTERBACH, RITTNER ANDREW, Jun., resigned April 9, 1934.

MOSELEY, DAVID LEIGH, Jun., resigned May 1, 1934.

NAPIER, LEROY MUNROE, JR., Jun., resigned April 30, 1934.

SIPPLE, WILLIAM REGINALD, Jun., resigned April 18, 1934.

WALLER, OSMAR LYSANDER, M., resigned April 24, 1934.

WOEHRLIN, GEORGE JOHN, Assoc. M., resigned May 7, 1934.

DEATHS

ACEVEDO, OCTAVIO AUGUSTO. Elected M., Nov. 11, 1929; died April 30, 1933.

BERLE, KORT. Elected Assoc. M., Dec. 6, 1899; M., Nov. 4, 1903; died May 5, 1934.

CLARK, JAMES HOPKINS. Elected Assoc. M., Nov. 27, 1917; died Jan. 4, 1934.

DUFFIES, EDWARD JOHN. Elected M., Jan. 3, 1906; died April 17, 1934.

FENNO, EDWARD KIMBALL. Elected Affiliate, Sept. 12, 1916; died April 2, 1934.

KIRBY, ISAAC HENRY. Elected Assoc. M., Nov. 1, 1905; died March 15, 1934.

LANGFITT, WILLIAM CAMPBELL. Elected M., June 6, 1921; died April 21, 1934.

LUDBERG, ANDREW PETER. Elected Assoc. M., Aug. 4, 1924; died April 11, 1934.

McCLAIN, JAMES BROWNSON. Elected Assoc. M., Sept. 2, 1914; died April 19, 1934.

PALMER, FREDERICK. Elected M., Oct. 4, 1899; date of death unknown.

PIERSON, ALBERT WARING. Elected Assoc. M., April 2, 1912; M., Aug. 12, 1920; died Mar. 15, 1933.

SHAW, GEORGE HARRY THORNTON. Elected M., Oct. 4, 1910; died April 17, 1934.

WRENTMORE, CLARENCE GEORGE. Elected Assoc. M., April 5, 1905; M., Oct. 5, 1909; died March 1, 1934.

Men Available

These items are from information furnished by the Engineering Societies Employment Service, with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fee is to be found on page 85 of the 1934 Year Book of the Society. To expedite publication, notices should be sent direct to the Employment Service, 31 West 39th Street, New York, N.Y. Employers should address replies to the key number, care of the New York Office, unless the word Chicago or San Francisco follows the key number, when the reply should be sent to the office designated.

CONSTRUCTION

CONCRETE TECHNICIAN; Assoc. M. Am. Soc. C.E.; technical education; married; 8 years construction experience, specializing in concrete design, control, charge of inspection force and concrete testing laboratory. Qualified to take charge of concrete field laboratory, concrete inspection force, and making special investigations in concrete and preparing reports. Location immaterial. Available one months notice. C-8022.

CIVIL ENGINEER: Assoc. M. Am. Soc. C.E.; 47; good health; married; license, New York; 9 1/2 years drafting, design, and estimating; 18 years field; all grades to, and including, division engineer. Principal engagements on water supply, hydro-electric, mill, and railroad work. Excellent record with former employers. Location immaterial. B-5199.

DESIGN

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 28; married; C.E. degree, Polytechnic Institute of Brooklyn; special course in structural engineering; 9 1/2 years practical experience in structural steel design for subways, piers, pier sheds, tunnels, buildings in railroad yards, and elevated structures. Checker of shop drawings; excellent draftsman; available immediately. Location anywhere. D-1754.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; Ph.D.; 7 years experience in the design of bridge and building structures; desires connection. C-3736.

CIVIL AND STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; licensed professional engineer; 30; B.S. in C.E. and C.E. degrees; experienced as a designer, draftsman, checker, detailer, and estimator of structural steel and reinforced concrete structures. References and samples of work sent on request. C-6186.

EXECUTIVE

CIVIL ENGINEER-ARCHITECT; M. Am. Soc. C.E.; registered engineer, Pennsylvania and New York. Registered architect, New York, New Jersey, Pennsylvania; 22 years design-construction, supervision, and maintenance of industrial plants, commercial, and some residential buildings. Department head, 10 years. Experienced in preparing and letting contracts and making reports, appraisals, studies, specifications. Available now. B-8117.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E., 44; married; experienced estimator, designer, all-round structural man, with 18 years experience in all phases of structural steel work. Graduate of University of Wisconsin. Available on short notice and will consider location anywhere for reasonable compensation. C-9851.

CIVIL AND CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; graduate; 34; single; New York State license; 14 years diversified experience, estimating, designing, supervising work, highways, subways, bridges, varied foundations, buildings, and water and sanitary works. Familiar with latest equipment in construction. Economical, efficient analyses of project. Speaks French, Russian, German. Available on short notice. Location immaterial. D-2918.

ENGINEER OR SUPERINTENDENT; M. Am. Soc. C.E.; 46; married; registered in California; specialist in building construction, high-class residences; efficient in office and field work; good executive; 20 years experience as superintendent of construction on America's finest homes and buildings. Speaks Spanish and Italian fluently. Available immediately. Location immaterial. D-2936-327-A-1 California.

CIVIL ENGINEER; M. Am. Soc. C.E.; 28 years experience covering investigations and reports, laboratory testing of materials, organization, design, reinforced concrete construction. Specialist in checking coast erosion by sea defence works based on sound and proved principle. Speaks French and Spanish. Location immaterial. Available now. D-2989.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; married; 35; graduate of University of Illinois; 12 years experience, including topography surveys, drafting, design of steel and reinforced concrete, sewers, sewage treatment, water works, hydraulics, airport and river terminal design; especially interested in sewerage and sewage treatment. Available immediately. Open for position in United States or foreign country. D-2941.

ENGINEER-MANAGER; M. Am. Soc. C.E.; technical training, Harvard University and Lowell Institute; New York and New Jersey licenses; 20 years on design, construction, and operation of water supply projects, highways, sewers, electric lighting plants, parks, buildings, bridges, tunnels, retaining walls, wharves, industrial and utility plants; desires position requiring broad range of experience and judgment. C-7633.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 35; graduate of University of Alabama; 15 years field and office experience on design and construction; now specializing in engineering and economic investigations; a successful organizer, administrator, and executive; minimum salary $4,000; location immaterial; now employed but will consider offer for engagement in near future. D-2943.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 33; married; graduate of Pennsylvania State College, 1923; New York State license; field and office experience, including investigations, reports, surveys, estimates, design, specifications, inspection, supervision on sewers, disposal plants, water supply, highways, railroad, small dams; 8 years as principal assistant to practicing engineer. Available immediately. B-9334.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; registered in Pennsylvania; married; 39; experience designing, directing design of highways and highway structures; 6 years in responsible charge of drafting room; concrete building design and construction experience; consulting engineer's office 6 years; experience water works, power plants, sewerage systems, preparation of reports. Desires engineering position, teaching municipal engineering, drafting, allied subjects. D-2942.

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